DESIGN OF DAMS—IRRIGATION

346 C

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IRRIGATION

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Design of Dams
Irrigation

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NOTE.—This book is made up of separate parts, or sections, as indicated by their titles, and the page numbers of each usually begin with 1. In this list of contents the titles of the parts are given in the order in which they appear in the book, and under each title is a full synopsis of the subjects treated.

DESIGN OF DAMS

IRRIGATION

DESIGN OF DAMS

Serial 2758 Edition 1

INTRODUCTION

GENERAL FEATURES OF DAMS

1. **Purpose of Dams.**—A dam is a barrier placed across a stream for the purpose of holding back the flow of water, thus forming a reservoir or storage basin. To accomplish this purpose, the dam must be impervious to the flow of water, or nearly so; and it must be so constructed that it will be able to resist the pressure of the water stored behind it. In order to prevent the water behind the dam from rising too high during flood stage, means must be provided for allowing the surplus water to escape. The usual manner in which such provision is made is by constructing part of the dam so that its top is below the high-water level. The water is thus allowed to flow over the low portion, which is known as the *spillway*. Sometimes a dam is constructed so that water can flow over the top for its entire length.

2. **Parts of a Dam.**—In Fig. 1 is shown a cross-section of a masonry dam. The exposed or down-stream surface *abcd* is generally known as the *face*, while the up-stream surface *efg* is usually called the *back*, or sometimes the *up-stream face*. The bottom *ae* of the dam is its *base*, the down-stream edge at *a* being the *toe* and the up-stream edge at *e* the *heel;* and the top *dg* is known as the *crest*. The height of the dam is the vertical distance *H* from the base to the crest. When the estimated normal high-water level is below the top of the dam, as at *i*, the vertical distance *ig* from the water surface to the crest is called the *superelevation* or *freeboard*.

The cross-section in Fig. 1 is for the bulkhead, or non-overflow part of the dam. At the spillway, the crest is generally shaped so as to facilitate the flow of water over it.

3. Usual Types of Dams.—The usual types of dams may be classified, according to the material of which they are built, as masonry, earth, rock-fill, and timber dams. Masonry dams may be further classified, according to the type of construction, as gravity dams, arch dams, and hollow dams. Various other types of dams have been designed and built, but, owing to the special nature of their construction, they will not be considered here.

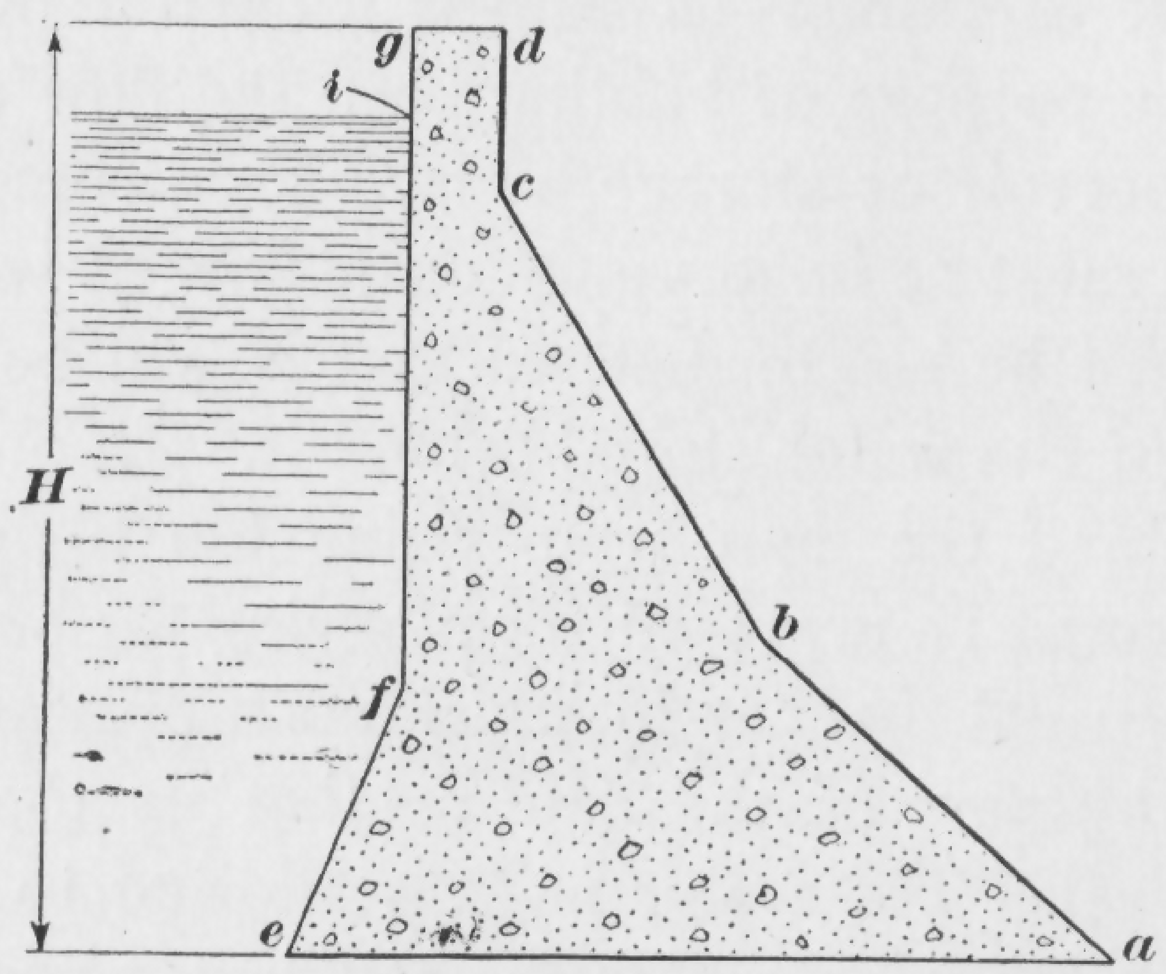

Fig. 1

4. Gravity Dams.—A gravity dam is one that is designed to resist by weight alone the forces exerted by the impounded water. If the dam consists of a solid mass of masonry and practically all of the required weight is provided by the material of which the dam is constructed, it may be termed a solid gravity dam. Some dams, which consist merely of a framework of masonry, are so built that the weight of the water resting on them is an important factor in their stability, and such dams are sometimes called hollow gravity dams. However, solid gravity dams are generally known simply as gravity dams and hollow gravity dams simply as hollow dams, and these simpler terms will be used here.

Most of the high masonry dams now in service are of the gravity type. Dams of this class are generally constructed of

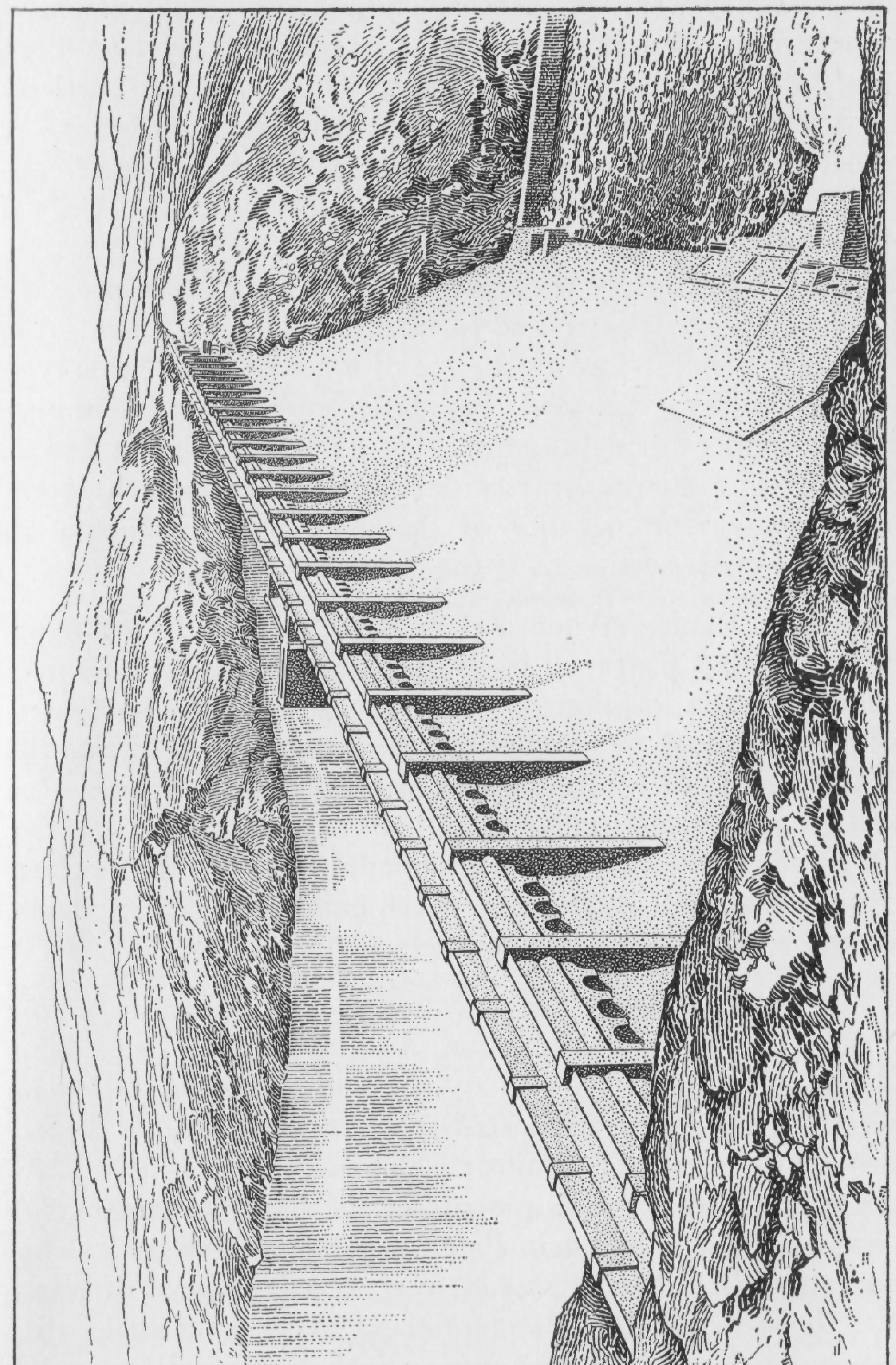

Fig. 2

ordinary mass, or plain, concrete. Formerly, rubble or cut-stone masonry was used extensively and rubble or cyclopean concrete was also popular, but these materials have been largely superseded by concrete in which the largest particles of aggregate are not more than about 3 inches in diameter. A typical gravity dam of plain concrete is shown in Fig. 2.

Gravity dams are usually quite wide at the base. Only a relatively small thickness of the masonry is needed for water-tightness, but a heavy mass is required to resist the pressure of the impounded water and to insure stability. Since great pressure is concentrated at the toe of a gravity dam, this type of dam requires a firm rock foundation. A gravity dam may be built either straight across the stream or on a curve. Although in a curved gravity dam there is some arch action that increases the stability of the dam, it is customary to design the cross-section as if the dam were straight.

5. Arch Dams.—When a dam is to be built in a narrow canyon that has firm rock walls on both sides, advantage may be taken of the location by constructing the dam so that it will act as an arch and will transmit the water pressure to the sides rather than the bottom of the canyon. The use of a much smaller cross-section than that required for a gravity dam of the same height is thus made possible, and a great saving in materials is effected. Arch dams, like gravity dams, are generally built of plain concrete.

6. Hollow Dams.—There are various types of hollow dams. The two main types may be classified as deck dams and multiple-arch dams. As usually constructed, a *deck dam* consists of a sloping reinforced-concrete deck slab or water-bearing member that is supported by a series of triangular-shaped buttresses or walls placed parallel with the stream. A typical deck dam is shown in Fig. 3, where a sloping deck slab *a* is supported by the buttresses *b*. To prevent seepage of water under the dam, the deck slab terminates in a cut-off wall *c* that extends into the foundation bed. Each buttress is provided with a footing *d* which distributes the load on the buttress over a sufficient area of the foundation bed. In order to brace the buttresses laterally, the reinforced struts *e* are usually provided. When the deck of a hollow dam consists of a series of reinforced-concrete arches tied rigidly to the buttresses, the dam is known as a *multiple-arch dam*. The spacing of the buttresses in hollow dams depends on the type of construction and the height of dam. It may range from 16 to 80 feet.

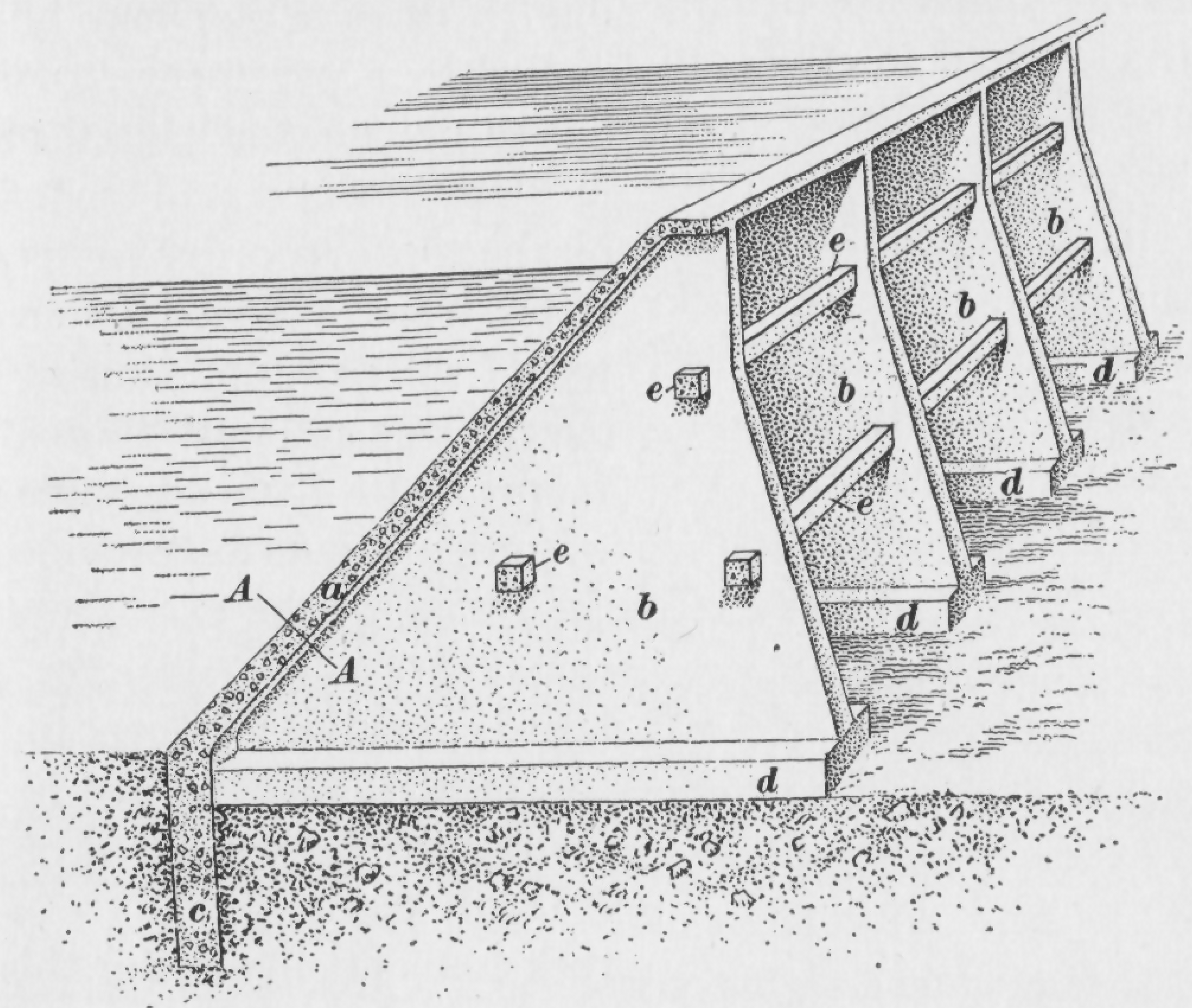

FIG. 3

Hollow dams may be constructed on comparatively soft foundation beds, as they are not very heavy. However, the footings *d* must be so proportioned that the pressure on the soil will not exceed its safe bearing capacity. On the softer varieties of foundation beds, a continuous floor, or mat, may be used instead of independent footings.

7. Earth Dams.—As shown in Fig. 4, an earth dam consists essentially of a simple embankment *a* of loam, sand, gravel, and clay, whose mass is more than sufficient to resist the pressure of the water. While the material is being deposited it is compacted so as to increase its impermeability. In most cases the water-tightness of the dam is further increased by means of an embedded curtain wall, or core wall, *b* of concrete or of

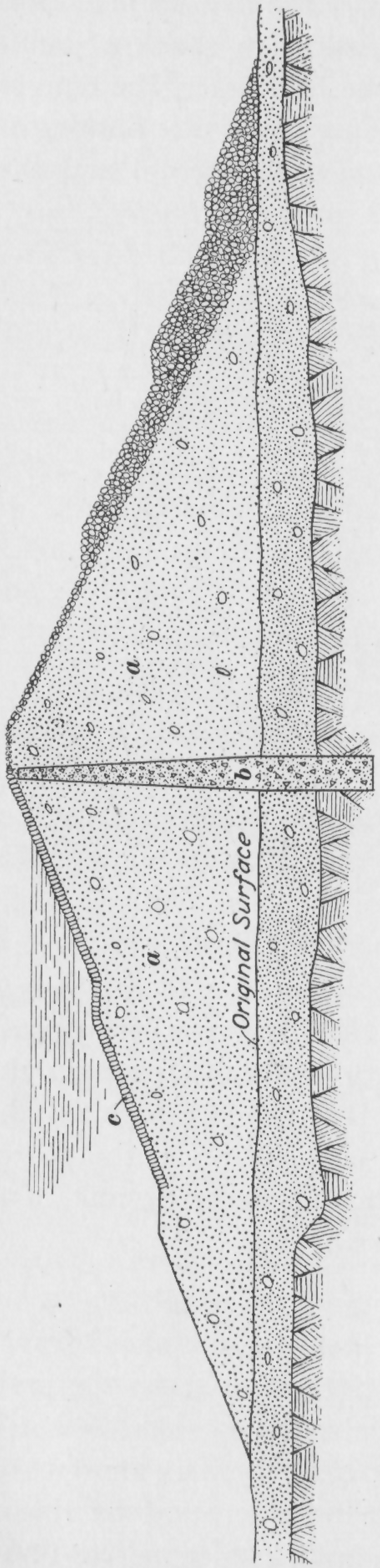

FIG. 4

puddle, which is a mixture of clay, sand, and gravel that is well moistened and rammed in place. Earth dams must also be protected against wave action and overflow. Erosion of the dam due to wave action may be prevented by a pavement *c* of stone blocks or concrete placed on the up-stream slope and sometimes on the top. In order to keep the water from ever rising above the top of the dam and causing serious damage and possibly complete failure of the dam, an ample spillway area should be provided for the escape of the excess water in times of flood. Also, the freeboard should be sufficient to allow for possible uncertainty in estimating the high-water level.

Earth dams do not require a rock foundation and, for this reason, are frequently constructed where a site suitable for a gravity dam of solid masonry is not available. When protected against overflow, seepage, and wave action, the earth dam is safe and durable, resisting the destructive forces of nature better, perhaps, than any other type of construction. Even earthquakes are not likely to cause serious damage, which may be a factor of considerable importance in some localities. Moreover, when satisfactory materials for an earth dam

are readily available, this type is often much cheaper than a masonry dam. However, earth dams are not generally suited for heights much in excess of 200 feet.

8. Rock-Fill Dams.—A cross-section of a typical rock-fill dam is shown in Fig. 5. Such a dam is of the same general type as an earth dam, but the embankment *a* is constructed of loose rock. Impermeability may be obtained in several different ways. Sometimes a rubble-masonry or reinforced-concrete facing is provided on the up-stream slope; or the slope is covered with timber planking fastened to wooden studs embedded in the rock or with steel plates secured to

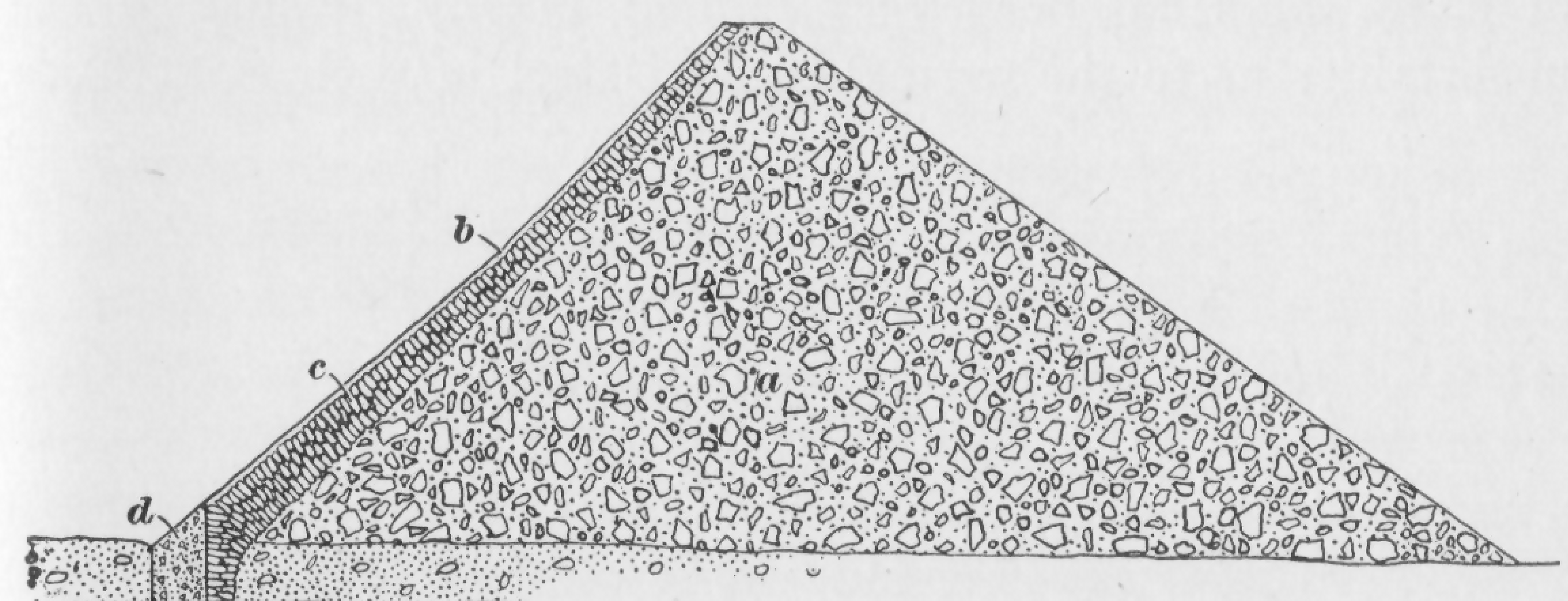

FIG. 5

I beams set in the rock. Less often, a center core wall of concrete or puddle is used instead of the facing. The up-stream face of the dam shown in the illustration is provided with a reinforced-concrete apron *b* which rests on a layer *c* of hand-placed rock and which terminates at the bottom in a concrete toe-wall *d*.

When large quantities of suitable rock are at hand, the cost of construction of a rock-fill dam may be very low, as compared with that of other types. However, the foundation must be substantial and the dam must be protected against overtopping by flood water.

9. Timber Dams.—Many timber dams have been built, but most of them are for relatively small projects where the heads are low. In the usual type of construction, which is illustrated in Fig. 6, wooden cribs are filled with stone to give the

dam weight and solidity, and the up-stream face is covered with a layer of planks and often also with an earth embankment. Although such dams are fairly cheap in first cost and are quite durable when properly built, they are not watertight and usually require considerable maintenance.

10. Steel Dams.—Steel has been much used in the construction of gates, shutters, and other devices that may be classed as movable dams. Also, as previously stated, steel plates have been placed on the up-stream face of rock-fill dams in order to provide a waterproof membrane. But, the employment of steel as the main material in the construction of fixed dams has been very limited, chiefly because of the uncertainty as to the permanence of steel in such structures.

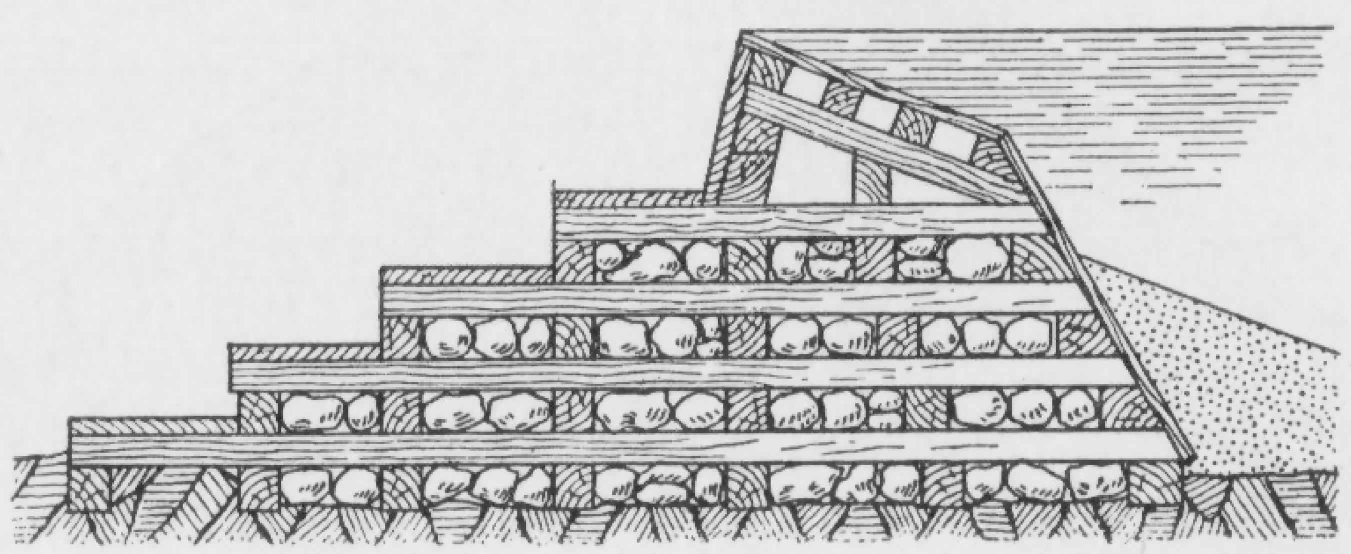

Fig. 6

11. Choice of Type of Dam.—The type of dam to be adopted in any particular case is governed mainly by the consideration of safety and the cost of construction and maintenance. Safety is usually of primary importance, because the failure of a dam may result in great property damage and also in loss of life. Thus, if an investigation of the foundation and other characteristics of the locality shows that certain types of dams could not be built with safety on the selected site, such types should not be given further consideration.

The important factors in determining the cost of construction are the required height and length of the dam, and the price and availability of the various materials that can be used. However, in deciding on the material, durability and maintenance must be taken into account. For a permanent structure, a comparatively expensive material that requires no

maintenance is often more economical in total cost than a cheaper material that must be repaired and replaced frequently.

INVESTIGATION OF SITE

12. Important Factors.—In most cases the dam itself is only one part of a complete project and its general location is determined by a consideration of the plan of the project as a whole. There will usually be several possible sites for the dam, each of which will require more or less thorough investigation. The principal items to be considered at a site, in addition to the safety of the dam and its required length and height, are the following: the character of the foundation, the available materials of construction, the transportation facilities and the accessibility of the site, the location of the construction camp, the capacity of the reservoir, the value of the land to be flooded, and the need for relocating highways or railroads. The velocity of the current is sometimes an important factor because it may determine the method of construction that is to be employed. Other factors that may influence the selection of the dam site are the location and construction features of the spillway and the location of the power house.

Before the site for the dam is finally selected it is necessary to make preliminary and final investigations. Actually, there is no sharp dividing line between the two kinds of investigations, as there are usually intermediate steps that belong partly to both.

13. Preliminary Investigations.—The main object of the preliminary investigations is to determine whether or not the project as a whole is feasible in the territory under consideration. At the outset in such investigations, it is usually necessary to make an extensive reconnaissance survey of the territory, chiefly for the purpose of locating the several possible sites for the dam and obtaining sufficient data for the preliminary estimates of cost. Each possible dam site should be examined carefully and enough notes should be taken to estimate the probable height and length of the dam and its

approximate cost, and also the area and value of the land to be flooded and the cost of the water rights. A study should be made of the surface indications at each dam site to determine the nature of the foundation materials and, if rock is to be expected, a rough estimate should be made of its character and of the probable depth of the overlying material. Samples of the soil to be excavated and of local materials that could be used for construction purposes should be collected for reference. Also, data should be gathered for estimating roughly the cost of clearing the reservoir site and of relocating the highways and railways that may be on it, and notes should be taken for determining the approximate location of the construction camp.

14. Final Investigations.—After the data obtained in the preliminary investigations have been studied and the less desirable possible locations of the dam have been eliminated, the next step is to investigate more thoroughly the remaining sites and to compare their advantages. Usually, the final choice of the dam site can be made without conducting a complete investigation at each of the possible sites, and it is only at the location finally selected that exact information concerning all the factors to be investigated is required.

The main purposes of the final investigations are to establish the exact location of the dam and its spillway; to secure accurate information concerning the character of the foundation at the dam site; to determine the required length and height of the dam and to obtain all data that may influence its design; to fix the limits of the land to be covered by water and of the watershed area; to determine the new locations for the railroads or highways that may be within the reservoir limits; to select locations for construction equipment and camp, and for construction railroads and highways; to determine the sources and amounts of local materials that are suitable for construction purposes; and to obtain sufficient information for making an accurate estimate of the cost of the entire project. In every case Government regulations must be ascertained and followed.

In the case of an important dam, the foundation bed should meet the following four requirements: (1) It should be strong enough to resist the maximum compressive stress to which it will be subjected; (2) it should be reasonably impervious to water; (3) it should not contain substances that will soften or dissolve in water or that will shrink or swell with changes in the water content; (4) its stability should not be affected by movements due to faults or earthquakes. In order to obtain the data needed in the final investigation of the foundation at the site, test borings are made over its entire surface, and the characteristics of the various strata and their depths are determined at each point.

A geologist is usually engaged to make a complete report regarding the geological features of the site. His report should contain information concerning the water-tightness of the reservoir basin, the character of the soil or rock formations at the dam site, the facilities for providing the spillway, the quality and amount of construction materials that are available locally, and the possibilities of silt being deposited in the reservoir. If a tunnel is to be used to divert the water during construction or to provide an outlet for the impounded water, the geologist will have to report on the suitability of the rock along the proposed route of the tunnel. Although the rock at a dam site may be amply strong to resist crushing under the pressure exerted by the dam, it may contain substances that are soluble in water or there may be sources of weakness, such as faults, joints, bedding planes, or seams of clay. The water from behind the dam, being under great pressure, is likely to enter the joints or seams, causing the rock to slide and the dam on it to fail. It is also important that the foundation bed on which a masonry dam rests should be quite uniform throughout the entire bearing area, in order to prevent unequal settlement of different parts of the dam and consequent cracking of the masonry. When it is planned to construct the spillway in the natural rock, the resistance to wear and erosion must be considered.

The final estimate of the cost of the entire project is made after all necessary information has been obtained. It includes

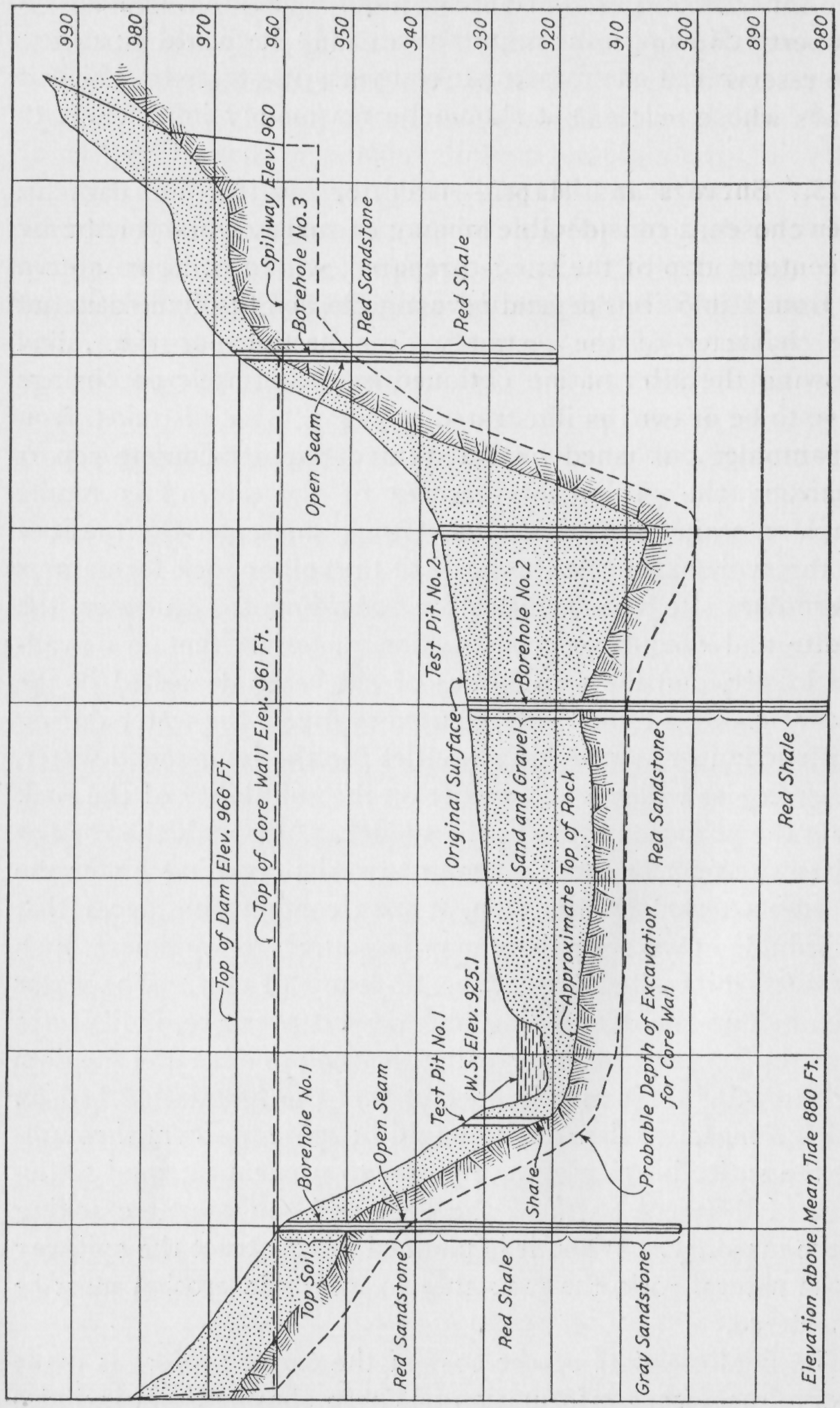

FIG. 7

not only the cost of constructing the dam but also the cost of property damage necessitated by raising the water surface in the reservoir, and the cost of relocating the highways or railroads whose original routes would be flooded.

15. Surveys and Maps.—After the site for the dam has been chosen, a considerable amount of survey work is required. A contour map of the site is prepared, with a contour interval of from 1 to 5 feet, depending upon the height of the dam and the character of the terrain. Cross-sections of the valley, showing the information obtained by soundings and borings, have to be drawn, as illustrated in Fig. 7, which is taken from a pamphlet published by the Water Supply Commission of Pennsylvania. It is also necessary to prepare topographic maps on which are shown the original position and condition of the stream, and the important features of the land to be overflowed, such as property lines, buildings, and roads. In addition, there will be surveys for possible routes of penstocks, pipe lines, canals, or other structures, and for the relocation of highways and railroad lines.

Photographs of the site are frequently of much value in designing the dam and preparing the foundation. They also furnish valuable records of conditions before construction began. The entire set of maps and photographs should be incorporated in a report of the final investigation.

GRAVITY DAMS

STABILITY

FORCES ACTING ON DAMS

16. External Forces.—The principal external forces acting on gravity dams are the pressure of the water held behind the dam, and the weight of the masonry. The water pressure on the back of the dam tends to move it in two ways: (1) The horizontal component of the pressure tends to cause the dam to slide horizontally on its base or at any horizontal section. (2) The pressure tends to overturn the dam about its toe. Resistance to sliding and to overturning is provided mainly by the weight of the masonry. The principal force resisting sliding at any section is the frictional force, which is equal to the product of the weight of the dam above the section and the coefficient of friction for the materials in contact. Overturning is resisted by the moment of the weight of the masonry about the toe of the dam.

Other forces that may increase the effect of the water pressure in tending to cause failure of the dam are pressure from ice that may form behind the dam, or from silt that may be deposited in back of the dam; the impact from waves; and wind pressure. Also, where water seeps under the dam, there is an uplift pressure which reduces the effective weight of the dam and, therefore, decreases its resistance to sliding and overturning. On the other hand, where there is a lower pool at the down-stream face, the pressure of the water in the pool against that face helps resist overturning.

In the investigation or design of a gravity dam, it is customary to consider the forces acting on a strip of the dam 1 foot long, that is, on a slice of the dam between two cross-sections 1 foot apart. The volume of masonry in the strip, in cubic feet, is then numerically equal to the area of either cross-section, in square feet. In other words, areas in square feet

represent volumes in cubic feet. Also, the other forces that are to be considered are those acting on a strip of dam of unit length and, as the length of the dam may be disregarded, the computations are greatly facilitated.

When it is expected that water will flow over the top of the dam, special measures must be taken to prevent the formation of a partial vacuum on the face of the dam. If such a vacuum is created, the difference between the actual pressure on the face and the atmospheric pressure causes a thrust on the dam. Moreover, the amount of the thrust may fluctuate quite rapidly as the conditions of flow vary, and, as a result, there may be produced a vibration or trembling of the dam which is much more serious than the effect of a constant thrust.

17. Weight of Masonry.—The weight of concrete depends on the kind of aggregate used. For the materials commonly employed in dams where heavy concrete is desirable, the weight will usually be between 145 and 155 pounds per cubic foot. Stone masonry weighs from 140 to 165 pounds per cubic foot, depending on the materials and the type of construction. In designing a dam, it is always advisable to determine by actual test the weight of the particular masonry to be used. In all problems that follow, the weight of masonry will be assumed as 150 pounds per cubic foot.

18. Crushing Stresses in Dam and Foundation.—The weight of the dam, the overturning effect of the water pressure, and other forces produce a pressure on the foundation. Therefore, in order to prevent excessive settlement of the structure, the base must be wide enough to insure that the maximum pressure will be below the safe bearing capacity of the foundation bed. Also, the material of the dam must be strong enough to resist the crushing stresses to which it may be subjected.

19. Provision for Dissipation of Heat.—In building a massive concrete structure, such as a high gravity dam, consideration must be given to the heat generated in the mass by the chemical reactions that take place during the hardening of

the concrete. Unless some provision is made for controlling the heat, it may not be entirely dissipated for a number of years after the structure is completed, and shrinkage cracks will continue to form as long as the mass continues to cool and contract. Obviously, such a condition is undesirable in a dam, where water-tightness is so important.

There are several methods for controlling the temperature of the concrete: (1) The dam may be constructed of precast concrete blocks of moderate size that have been cooled to a suitable temperature before they are laid in the structure. (2) The concrete for the dam may be poured in place so as to form isolated blocks with radial and longitudinal slots between adjacent blocks; after the heat has been sufficiently dissipated, these slots are filled with grout. (3) Use may be made of special cements that generate less heat than ordinary portland cement. (4) A cooling system may be installed in the concrete; thus, a system of small pipes is embedded in the concrete and, after the concrete has been in place for several days, cool water is forced through the pipes until the temperature of the mass has been lowered to normal. One or more of these methods may be used in the construction of large dams. In the Boulder dam, for which the maximum height is 732 feet and the corresponding width of base is 650 feet, provision for heat control has been made by means of all but the first of the methods here described.

WATER PRESSURE

20. General Formulas for Water Pressure.—When a surface is submerged in water, the unit water pressure at any point of the surface may be found by the relation

$$p=62.5\,h_x \qquad (1)$$

in which p = unit pressure, in pounds per square foot, acting at right angles to surface;

h_x = head, in feet, on point at which pressure is required.

For instance, at a point on the back of a dam 20 feet vertically below the water surface, the unit pressure is $p=62.5\times 20$

= 1,250 pounds per square foot. This pressure always acts at right angles, or normal, to the surface.

The total normal pressure on any plane surface immersed in water is found by the formula

$$P=62.5\,a\,h_c \qquad (2)$$

in which P = normal pressure on entire surface, in pounds;

a = area of surface, in square feet;

h_c = head on center of gravity of surface, in feet.

21. Water Pressure on Dam With Vertical Back.—When the back of the dam is vertical, as in Fig. 8 (a), the area of each vertical strip 1 foot long that is below the surface of the water

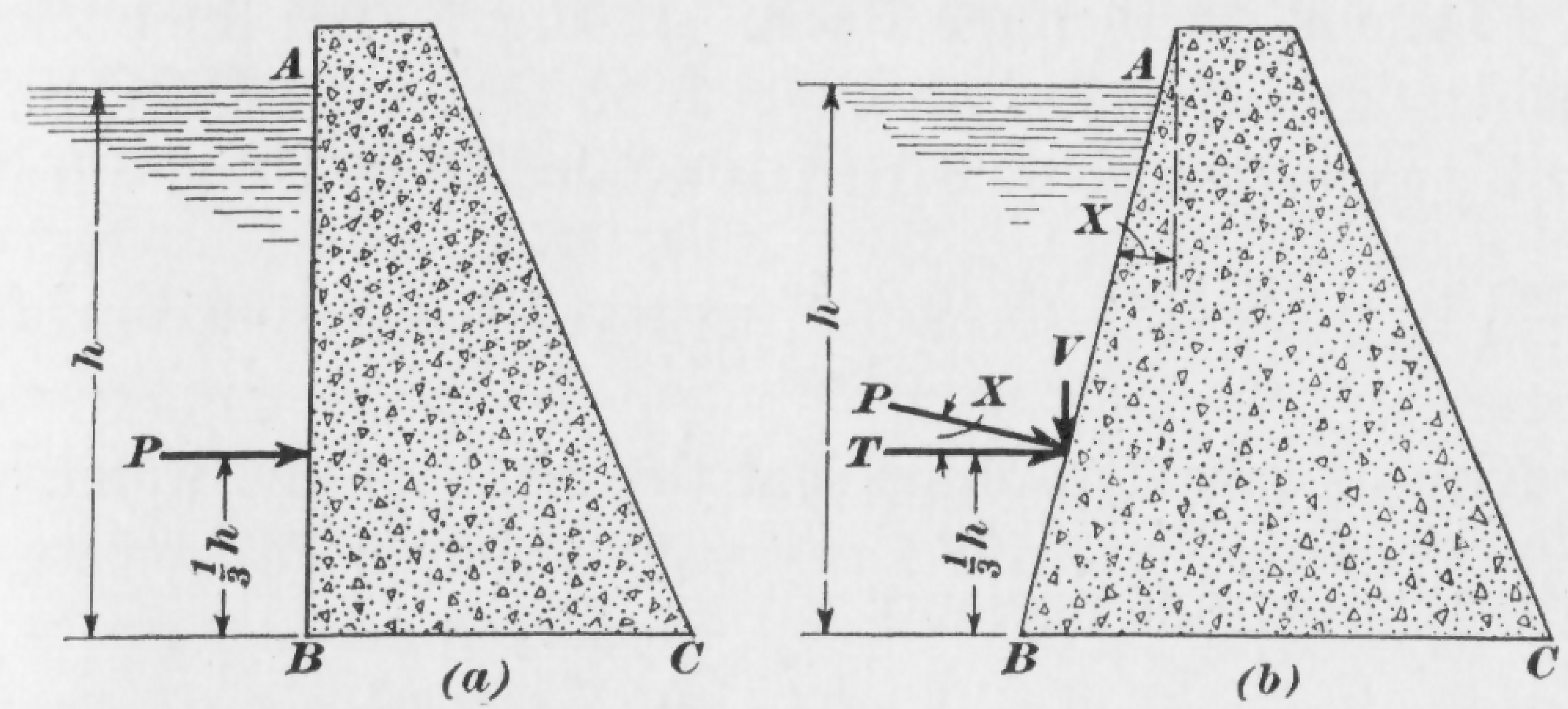

FIG. 8

is equal to the depth of the water, or h. Since the head on the center of gravity of this area is $\frac{h}{2}$, the total normal pressure on the back of the dam is, by formula 2 of the preceding article, $P=62.5\times h\times\frac{h}{2}$ or

$$P=31.25\,h^2 \qquad (1)$$

In this case, the pressure P, being at right angles to the vertical back AB, is horizontal.

The center of pressure, or the point on the back of the dam at which the resultant pressure may be assumed to be applied as a concentrated load, is at a vertical distance above the base of the dam equal to one-third of the depth of the water, that is,

$\frac{1}{3}h$ in Fig. 8 (*a*). Hence, the moment of the water pressure about the toe C of the dam is $M = 31.25\,h^2 \times \frac{1}{3}h$ or

$$M = 10.42\,h^3 \qquad (2)$$

EXAMPLE.—If the depth of the water behind the dam represented in Fig. 8 (*a*) is 53 feet 6 inches, find (*a*) the horizontal thrust per foot of dam due to the water pressure and (*b*) the moment of the pressure about the toe of the dam.

SOLUTION.—(*a*) Here, $h = 53$ ft. 6 in. $= 53.5$ ft. Then, by formula 1, the required thrust is

$$P = 31.25\,h^2 = 31.25 \times 53.5^2 = 89{,}400 \text{ lb. Ans.}$$

(*b*) The required moment is, by formula 2,

$$M = 10.42\,h^3 = 10.42 \times 53.5^3 = 1{,}596{,}000 \text{ ft.-lb. Ans.}$$

22. Dam With Inclined Back.—In Fig. 8 (*b*), the back AB of the dam is inclined at an angle X to the vertical. Here the area of a strip 1 foot long that is subjected to water pressure is equal to the length of AB or $\frac{h}{\cos X}$, the head on the center of gravity of this strip is half of the depth of the water or $\frac{h}{2}$, and the total normal pressure is

$$P = 62.5 \times \frac{h}{\cos X} \times \frac{h}{2} = \frac{31.25\,h^2}{\cos X}$$

This pressure is at right angles to the sloping back of the dam and therefore acts at an angle X to the horizontal. However, in considering the effect of the water pressure on the dam, it is usually convenient to resolve the pressure into horizontal and vertical components. The horizontal component is $T = \frac{31.25\,h^2}{\cos X} \times \cos X$, or

$$T = 31.25\,h^2 \qquad (1)$$

which is the same as the horizontal pressure on a dam of the same height whose back is vertical. The vertical component of the pressure on the inclined back is $V = \frac{31.25\,h^2}{\cos X} \times \sin X$, or

$$V = 31.25\,h^2 \tan X \qquad (2)$$

In this case, also, the center of pressure is at a distance above the base of the dam equal to $\frac{1}{3}h$ and the moment of the horizontal component of the pressure about the toe C is $10.42\,h^3$. The vertical component V assists the weight of the dam in resisting sliding and overturning.

23. When the back of the dam has different slopes at different depths below the water surface, as in Fig. 9, the horizontal component T of the resultant pressure on the entire dam has the same amount and the same line of action as if the back of the dam were vertical. However, the vertical component of the pressure on each portion of the back must be considered separately. If the top portion AB is vertical, there is no vertical pressure on it. The vertical component V_1 on the portion BC is equal to the weight of water in the trapezoid $ABCD$ and its line of action may usually be assumed to lie midway between AB and CD even though the center of gravity of that trapezoid is a little nearer to CD. Similarly, the vertical component V_2 on the portion CE is equal to the weight of the water in the trapezoid $CDFE$ and its line of action may be assumed to be midway between CD and EF.

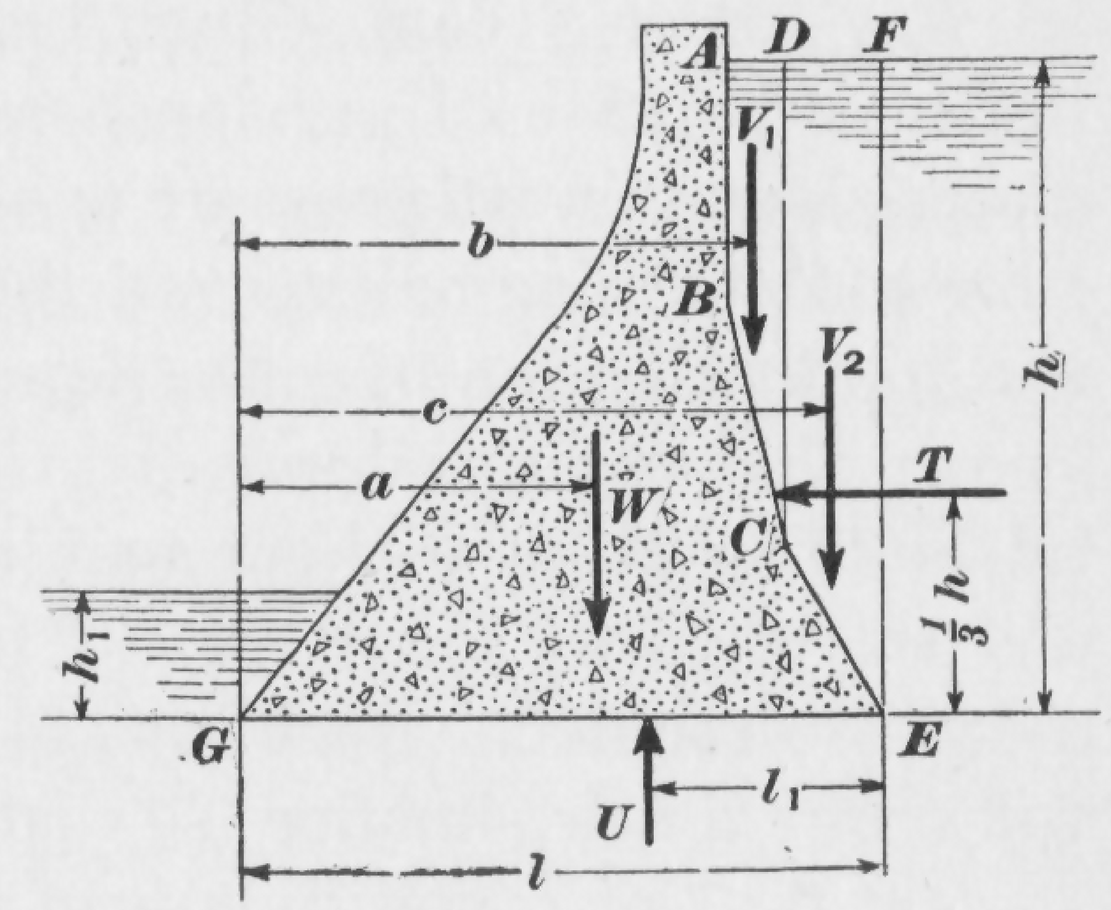

FIG. 9

If desired, the horizontal and vertical components of the pressure of the tailwater of depth h_1 against the down-stream face of the dam may be found by the methods of the preceding article, but this pressure is usually neglected, as it is not large and the practice is on the side of safety.

EXAMPLE.—In a dam like that shown in cross-section in Fig. 9, the depth h of the water is 130 feet, the depth to point C is 90 feet, and the horizontal distance from C to E is 20 feet. Find (*a*) the resultant horizontal thrust on a 1-foot strip of the back of the dam and (*b*) the amount of the vertical component of the pressure on the portion CE.

SOLUTION.—(*a*) The required horizontal thrust is

$$T = 31.25\,h^2 = 31.25 \times 130^2 = 528{,}000 \text{ lb. Ans.}$$

(*b*) The area of the trapezoid *CDFE* is

$$DF \times \frac{CD+EF}{2} = 20 \times \frac{90+130}{2} = 2{,}200 \text{ sq. ft.}$$

For a strip of dam 1 ft. long, the volume of water is that of the prism whose base is the trapezoid and whose altitude is 1 ft. This volume is 2,200 cu. ft. and the weight of the water is 2,200×62.5=137,500 lb. Ans.

UPLIFT

24. Cause of Uplift.—When water from a reservoir works its way under a dam, either because of defects in the dam itself, or because of perviousness or faults in the foundation material, an upward pressure or uplift is created. It is impossible to determine beforehand the exact amount of this uplift and its distribution over the foundation. However, it is well known that many failures of gravity dams have been caused by building them without making any allowance for uplift, and therefore the effect of uplift should be considered when a dam is designed. Each type of dam and each particular site will create a new situation, so that allowance for uplift requires a great deal of sound judgment as well as knowledge of the facts concerning the site under consideration and a comparison with other dams constructed under similar conditions.

25. Reduction of Uplift.—The amount of uplift in any particular case depends mainly on the care taken in construction to prevent water from seeping under the dam. Where the foundation is solid bed-rock without faults or cracks, the uplift will be so small that it can be disregarded. However, the uplift force increases as the foundation becomes more permeable. The best method of reducing the effect of uplift depends to a great extent on the character of the foundation material. In rock containing seams, a method of minimizing seepage is to seal the cavities by drilling holes in the rock and forcing cement grout into them under pressure. The uplift pressure may also be reduced by inclining or stepping the construction joints upwards in the down-stream direction.

In porous material, a cut-off wall at the heel of the dam, which is constructed by excavating a trench in the foundation down to solid impervious rock and filling it with dense concrete, is often effective. Some recent dams have been provided with drains, which collect the water that percolates through the foundation and convey it to the down-stream face where it is discharged without causing uplift. Where it is desired to prevent water from entering the masonry through the up-stream face of the dam, that face may be coated with gunite. However, as none of these measures can be relied on to eliminate uplift entirely, some allowance for uplift is usually made.

26. Allowance for Uplift.—When water flows beneath the dam without encountering an impermeable obstruction, the unit pressure causing uplift may be assumed to decrease at a uniform rate from the heel of the dam to the toe. At the heel, the unit pressure is that due to the head of water behind the dam; thus, in Fig. 9, it is 62.5 h pounds per square foot. Also, at the toe, the unit pressure is that due to the head of water below the dam, as the head h_1, Fig. 9. If the entire base of the dam were subjected to uplift, and the width of the base is l, then the total pressure would be $62.5\,l \times \frac{h+h_1}{2}$. However, in the case of an actual dam, the water would not have access to all parts of the area of the base. Hence, the total uplift per linear foot of dam may be determined by the formula

$$U = 31.25\,k\,l(h+h_1) \qquad (1)$$

in which U = total uplift, in pounds;

k = decimal part of width of base that is not in absolute contact with foundation;

l = width of base, in feet;

h = head of water at up-stream face of dam, in feet;

h_1 = head of water in pool at down-stream face of dam, in feet.

In many important dams, the value of k was taken as .67, and this seems to be the greatest value that need ever be used. However, a smaller value is generally considered sufficient and

in the problems that follow, k will be assumed as .5. The horizontal distance l_1 from the heel of the dam to the point at which the uplift may be assumed to be concentrated is the same as the distance from the longer base to the center of gravity of a trapezoid whose bases are h and h_1 and whose altitude is l. Thus,

$$l_1=\frac{l}{3}\times\frac{h+2\,h_1}{h+h_1} \qquad (2)$$

When an impervious cut-off wall is constructed in the foundation at the heel of the dam, it is more reasonable to assume that the unit pressure due to the head at the toe will act over the entire exposed base of the dam. Then the formula for the total uplift is

$$U=62.5\,k\,l\,h_1 \qquad (3)$$

Unless the dam itself is constructed of dense, impervious material, provision for uplift should be made in a similar manner at each horizontal section of the dam, the head h at any section being the depth from the water surface to the level of the section.

EXAMPLE 1.—(a) A dam is 96 feet wide at the base. If the water stands at a height of 120 feet above the base at the heel and 36 feet at the toe, what is the total uplift on the base? (b) Find the horizontal distance from the heel to the point of application of the resultant uplift.

SOLUTION.—(a) In formula 1, k is assumed to be .5, l=96 ft., h=120 ft., and h_1=36 ft. Therefore,

$$U=31.25\,k\,l(h+h_1)=31.25\times.5\times96\times(120+36)=234{,}000 \text{ lb. Ans.}$$

(b) By formula 2,

$$l_1=\frac{l}{3}\times\frac{h+2\,h_1}{h+h_1}=\frac{96}{3}\times\frac{120+2\times36}{120+36}=39.4 \text{ ft. Ans.}$$

EXAMPLE 2.—If the width of the dam in the preceding example is 42 feet at a depth of 60 feet below the upper water level, what is the total uplift at that depth?

SOLUTION.—In this case, the section under consideration is above the lower water level. Hence, the values to be substituted in formula 1 are: l=42 ft., h=60 ft., and h_1=0. Thus,

$$U=31.25\,k\,l(h+h_1)=31.25\times.5\times42\times60=39{,}400 \text{ lb. Ans.}$$

EXAMPLES FOR PRACTICE

1. A dam with a vertical back impounds water to a depth of 48 feet. (a) Calculate the resultant water pressure per foot of dam. (b) What is the direction of that pressure? Ans. $\begin{cases}(a)\ 72{,}000\text{ lb.}\\(b)\ \text{Horizontal}\end{cases}$

2. (a) At what distance below the water surface is the resultant pressure applied in the preceding example? (b) What is the moment of the pressure about the toe of the dam? Ans. $\begin{cases}(a)\ 32\text{ ft.}\\(b)\ 1{,}152{,}000\text{ ft.-lb.}\end{cases}$

3. The back of a dam slopes at the rate of 1 horizontal to 6 vertical. If the depth of the water is 72 ft., find (a) the horizontal component and (b) the vertical component of the water pressure per linear foot of dam. Ans. $\begin{cases}(a)\ 162{,}000\text{ lb.}\\(b)\ 27{,}000\text{ lb.}\end{cases}$

4. Determine the vertical pressure per foot of dam on the portion BC, Fig. 9, if the depths from the water surface to the points B and C are, respectively, 42 and 80 feet, and the horizontal distance from B to C is 5 feet. Ans. 19,060 lb.

5. The base of a dam is 80 feet wide. At the heel the depth of the water is 104 feet and at the toe it is 28 feet. Find (a) the total uplift per foot of dam, and (b) the distance from the heel of the dam to the point of application of the resultant uplift. Ans. $\begin{cases}(a)\ 165{,}000\text{ lb.}\\(b)\ 32.32\text{ ft.}\end{cases}$

OTHER EXTERNAL FORCES

27. Earth Pressure.—All rivers and streams carry earth in suspension, the amount depending on the velocity of flow of the water. When a dam is built across a stream, the velocity of the water is reduced near the obstruction and, as a result, a layer of silt is deposited behind the dam. This layer often attains a considerable depth unless provision is made for reducing it by installing sluices in the lower part of the dam and flushing periodically. Occasionally earth is deposited also against the down-stream face of the dam. The horizontal pressure exerted on either face of the dam by the deposited material may be found by the formula

$$P'=\frac{wh_2^2}{2}\left(\frac{1-\sin Z}{1+\sin Z}\right)$$

in which P' = total horizontal pressure, in pounds;

w = effective weight of deposited material, in pounds per cubic foot;

h_2 = depth of material at dam, in feet;

Z = angle of repose of material.

The resultant pressure P' is applied at a distance $\frac{2}{3}h_2$ below the surface of the deposit. For the usual conditions the effective weight of the deposited material when submerged is practically the same as that of water or 62.5 pounds per cubic foot. The angle Z varies from 0° for liquid mud to 30° for sand and gravel.

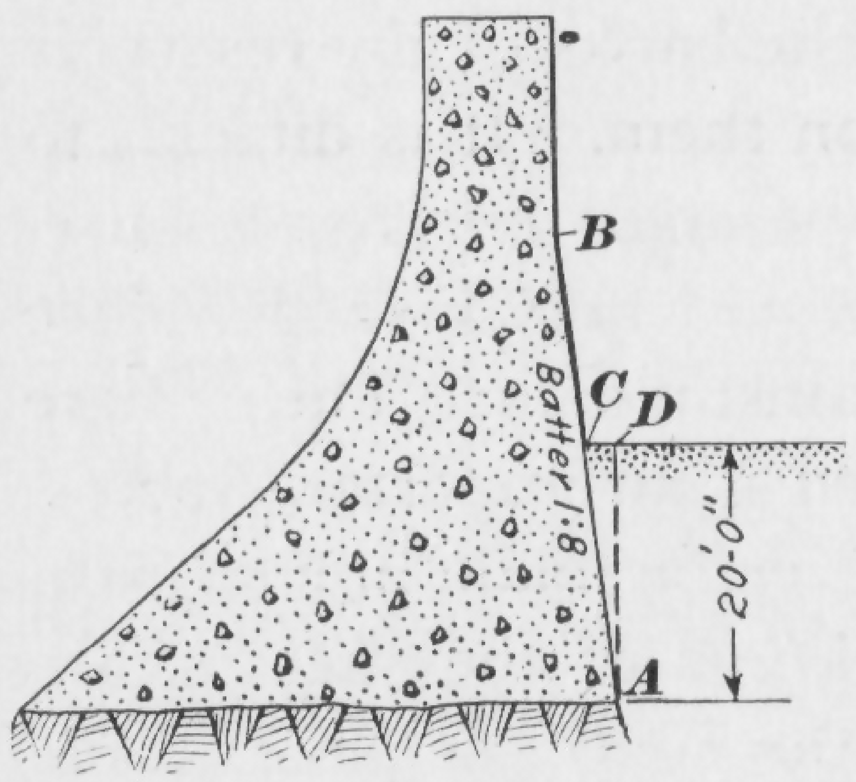

Fig. 10

Where the surface of the dam is inclined, the material deposited against it also exerts a vertical pressure equal to the effective weight of the deposit directly above the dam.

Example.—The cross-section of a dam is shown in Fig. 10, the batter of the back AB being 1 horizontal to 8 vertical. If the depth of the earth deposit is 20 feet, its effective weight is taken as 65 pounds per cubic foot, and the angle of repose is 30°, find (*a*) the horizontal pressure exerted on the dam by the deposited material and (*b*) the vertical pressure.

Solution.—(*a*) In this case, the horizontal earth pressure is

$$P'=\frac{wh_2^2}{2}\left(\frac{1-\sin Z}{1+\sin Z}\right)=\frac{65\times 20^2}{2}\times\left(\frac{1-\sin 30^\circ}{1+\sin 30^\circ}\right)=4{,}330 \text{ lb. Ans.}$$

(*b*) In Fig. 10 the distance CD is $\frac{1}{8}\times 20=2.5$ ft. and the area of the triangle ACD is $\frac{1}{2}\times 20\times 2.5=25$ sq. ft. The effective weight of earth in that triangle per linear foot of dam is $25\times 65=1{,}625$ lb., which is the required vertical pressure. Ans.

The pressure exerted by the deposited material acts in addition to the water pressure. However, owing to the fact that nearly all deposits of silt are quite impervious to water, it is customary to assume that silt pressure and uplift pressure due to the head of water behind the dam cannot act at the same time. Therefore, the common practice in designing dams is

to consider either uplift or silt pressure, the pressure used being the one for which the larger cross-section is required.

28. Ice Pressure.—Ice, like other solids, expands or contracts with a rise or fall in its temperature. As a result of a sharp drop in temperature, cracks may be formed in the sheet of ice at the surface of a reservoir, and the water in those cracks may freeze again. When the temperature then rises, the sheet expands and may exert a considerable pressure against the dam, unless the slopes of the banks of the reservoir are such that the ice is free to slide on them. It is difficult to determine the allowance that should be made for ice pressure, as in all probability a sheet of ice would buckle long before it could exert its maximum thrust against a dam. The climate in the region where the dam is located is an important factor. No allowance for ice pressure need ever be made in comparatively warm climates where the thickness of the sheet of ice will seldom exceed 6 inches, as in the southern part of the United States. An allowance as high as 47,000 pounds per linear foot of dam has been made in some cases. Often, the level of the water in the winter is so low that the dam is amply safe in resisting whatever ice pressure may be exerted at the lower level. Even where ice pressure is considered, it is assumed to be applied at a certain distance below the normal high-water level.

29. Wind and Wave Pressure.—As a rule, both wind pressure and wave impact are neglected in designing a dam. Wind pressure is important only when the water level is low, but its amount is much less than the pressure exerted by the water when the reservoir is full. Except in the case of a very small dam for an unusually large reservoir, the impact effect of waves is insignificant in comparison with other forces. However, the height of waves is sometimes a factor in choosing the height of freeboard on the dam.

UPWARD REACTION

30. Distribution of Pressure.—Any horizontal section of a dam is subjected to an upward reaction from the part of the structure below the section under consideration or from the foundation bed. When the resultant of the forces acting above the section passes through the center of gravity of the section, the upward unit pressure may be assumed to be uniformly distributed. However, for the most dangerous condition of loading that is considered in the design of a dam, the resultant cuts the section between the center and the face of the dam. For this condition, the unit pressure varies from a maximum at the face to a minimum at the back. These limiting pressures may be found by the following formulas:

$$p=\frac{R_v}{b}\left(1+\frac{6e}{b}\right) \qquad (1)$$

$$p_1=\frac{R_v}{b}\left(1-\frac{6e}{b}\right) \qquad (2)$$

in which p = maximum vertical unit pressure at face, in pounds per square foot;

R_v = vertical component of resultant force acting on dam, in pounds;

b = width of section, in feet;

e = distance, in feet, from center of section to point of application of resultant;

p_1 = minimum vertical unit pressure at back, in pounds per square foot.

31. Rule of Middle Third.—If the value of p_1 in formula 2 of the preceding article were negative, there would be a tensile stress in the masonry at the back of the dam and a tendency for the dam to rise at the heel. As a result openings would be started at joints where the water could readily enter. The uplift pressure thus produced would be much greater than that usually considered in design. Hence, the joint would be widened and, as the effective bearing width of the joint would be reduced, the point of application of the resultant force would move farther toward the face of the dam. This progressive action could finally cause failure of the structure either by overturning or by sliding.

An examination of formula 2 shows that p_1 becomes zero when e is equal to $\frac{1}{6}b$, that is, when the resultant force is applied at the edge of the middle third of the base of the dam. Therefore, it is a fundamental principle in the design of a dam to provide a cross-section of such dimensions that the resultant of the forces acting above any horizontal section will cut that section within the middle third.

STABILITY AGAINST OVERTURNING

32. Requirements for Stability.—In the usual design of a dam, such as that shown in Fig. 9, it is assumed that the principal force tending to overturn the dam about its toe G is the horizontal component T of the water pressure. Other forces that are sometimes considered are the pressure of ice and that of silt deposited in back of the dam. The main resistance to overturning is assumed to be provided by the weight W of the dam. However, where an allowance for uplift is made, the uplift pressure U is assumed to reduce the effective weight of the dam and thereby to diminish the resistance to overturning. On the other hand, when the back of the dam or a part of it is inclined, the vertical component of the water pressure on each inclined portion, such as V_1 or V_2, helps to resist overturning. In the case of the dam shown in cross-section in Fig. 9, the moment about the toe G of the force tending to overturn the dam is the product of the horizontal component T of the water pressure and its lever arm $\frac{1}{3}h$, or the value of the overturning moment M_o is

$$M_o=\tfrac{1}{3}\,T\,h$$

The moment about G of the forces tending to resist overturning, or the resisting moment M_r, is

$$M_r=Wa-U(l-l_1)+V_1b+V_2c$$

Theoretically, a dam is safe against overturning when the resisting moment, or the algebraic sum of the moments of the vertical forces and components, about the toe is greater than

the overturning moment, or the sum of the moments of the horizontal forces and components, about the toe. This condition is satisfied when the resultant of all forces acting on a dam cuts the base in back of the toe. However, the forces that tend to overturn the dam are sometimes liable to exceed

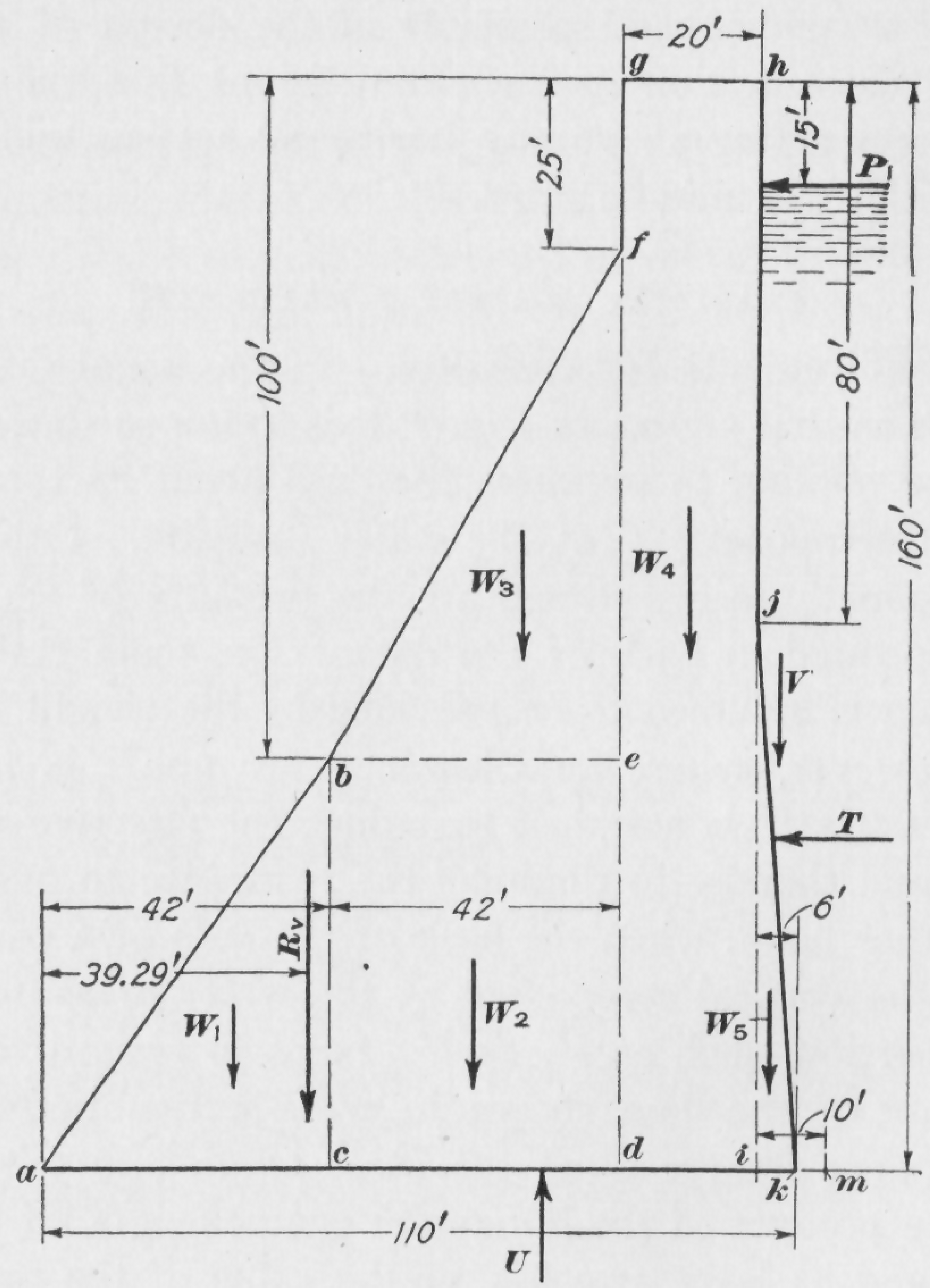

FIG. 11

the maximum values estimated for them. Also, the soil at the toe may yield somewhat under the great pressure produced there and the point about which overturning tends to take place will thus be shifted to some distance in back of the toe. Therefore, in order to safeguard against overturning, the factor of safety, or the ratio of the resisting moment to the overturning moment, should be about 2. In the design of a gravity dam, it is customary to consider the dam safe against

overturning when the resultant force cuts the base of the dam within the middle third. For this position of the resultant, tension in the masonry is also avoided.

33. Position of Resultant.—If the resultant force is resolved into horizontal and vertical components at the point where it cuts the base of the dam, the horizontal component passes through the toe and its moment about that point is zero. Hence, the moment of the vertical component is equal to the moment of the resultant itself, and the distance from the toe of the dam to the point where the resultant cuts the base may be found by dividing the resultant moment by the vertical component of the resultant force.

EXAMPLE.—In Fig. 11 is shown a trial cross-section for a proposed masonry dam. It is assumed that there will be an ice pressure of 36,000 pounds per linear foot of dam acting at the water surface when that surface is 15 feet below the top of the dam. Earth pressure is disregarded. The head causing uplift is assumed to vary from the full depth of water at the up-stream face to zero at the down-stream face. Calculate the distance from the toe of the dam to the point where the resultant of all the forces cuts the base, and determine whether that point is within the middle third.

SOLUTION.—The forces to be considered in this case are water pressure, ice pressure, uplift, and weight of masonry. The total horizontal component of the water pressure for a head of 145 ft. is

$$T = 31.25\,h^2 = 31.25 \times 145^2 = 657{,}000 \text{ lb.}$$

and its moment about the toe of the dam is

$$M = 10.42\,h^3 = 10.42 \times 145^3 = 31{,}770{,}000 \text{ ft.-lb.}$$

The ice pressure P_i, which is horizontal, is 36,000 lb. and its moment is $36{,}000 \times 145 = 5{,}220{,}000$ ft.-lb.

The vertical component of the water pressure on the sloping part *jk* of the back is

$$V = 6 \times \left(\frac{65+145}{2}\right) \times 62.5 = 39{,}400 \text{ lb.}$$

Its arm is assumed to be $110 - \frac{6}{2} = 107$ ft. and its moment is $39{,}400 \times 107 = 4{,}216{,}000$ ft.-lb.

To find the weight of the dam and its moment, it is convenient to divide the section into the triangles *abc, bef,* and *ijk,* and the rectangles *bcde* and *dghi.* The values may be tabulated as follows:

Part	Area, in Sq. Ft.	Arm, in Ft.	Moment
abc	$\frac{1}{2}\times42\times60=1{,}260$	$\frac{2}{3}\times42=28$	35,300
bcde	$42\times60 \quad =2{,}520$	$42+\frac{1}{2}\times42=63$	158,800
bef	$\frac{1}{2}\times42\times75=1{,}580$	$42+\frac{2}{3}\times42=70$	110,600
dghi	$20\times160 \quad =3{,}200$	$84+\frac{1}{2}\times20=94$	300,800
ijk	$\frac{1}{2}\times6\times80 = \ 240$	$104+\frac{1}{3}\times6=106$	25,400
Totals	8,800		630,900

The total weight of a linear foot of the dam is 8,800×150=1,320,000 lb., and the moment of this weight about the toe of the dam is 630,900 ×150=94,640,000 ft.-lb.

In formulas 1 and 2, Art. 26, h_1 is zero. Therefore, the total uplift pressure is, by formula 1,

$$U=31.25\,k\,l\,h=31.25\times.5\times110\times145=249{,}000 \text{ lb.}$$

and the distance from the heel to its line of action is, by formula 2,

$$l_1=\frac{l}{3}=\frac{110}{3}=36.67 \text{ ft.}$$

The moment of the uplift pressure about the toe of the dam is 249,000 ×(110−36.67)=18,260,000 ft.-lb.

Hence, the vertical component of the resultant force acting on the dam, which is the algebraic sum of the weight of the dam, the vertical component of the water pressure, and the uplift, is 1,320,000+39,400 −249,000=1,110,000 lb. Also, the moment of the resultant about the toe of the dam is equal to the algebraic sum of the moments of all the forces; the moments of the weight of the dam and of the vertical component of the water pressure are positive, and the other moments are negative. Thus, the resultant moment is 94,640,000+4,216,000−31,770,000−5,220,000 −18,260,000=43,610,000 ft.-lb., and the required distance from the toe of the dam to the point where the resultant force cuts the base is 43,610,000÷1,110,000=39.29 ft. Ans.

Since the distance from the toe of the dam to the edge of the middle third of the base is $\frac{1}{3}\times110=36.67$ ft., the resultant cuts the base 2.62 ft. inside the edge of the middle third.

STABILITY AGAINST SLIDING

34. Sliding on Foundation.—The horizontal forces acting on the back of a dam tend to slide it on its foundation. This sliding is prevented by the frictional force that is developed between the masonry of the dam and the material on which it rests; by the adhesion of the masonry to the foundation; and, where the surface of the foundation is irregular, by the shearing strength of the foundation material. However, adhesion and shearing are neglected in design, their effect being considered merely as increasing the factor of safety, and the resistance to sliding on the foundation is taken as the product of the vertical component of the resultant force and the coefficient of friction of the dam on the foundation. Where the foundation bed is of rock, the coefficient of friction is ordinarily placed at from .60 to .75. For good material and careful work, .75 is generally considered safe enough, but due allowance must be made for defects in the rock such as seams. Since the coefficient of friction of rock on moist clay is only about .33, seams in rock are especially dangerous when they contain clay. Hence, before a high dam is designed, a geologist should investigate the site for the purpose of discovering the possibility of seams and ascertaining their character. The coefficient of friction can then be selected in accordance with local conditions.

In order to insure stability against sliding, it is customary to make the weight of the dam sufficient to develop a probable frictional resistance that is higher than the horizontal component of the resultant force. On good rock foundations, a factor of safety of about $1\frac{1}{2}$ is usually considered desirable; that is, the product of the vertical component of the resultant force and the coefficient of friction should be at least $1\frac{1}{2}$ times the horizontal component of the resultant force. However, some authorities maintain that a dam founded on rock is stable when the frictional resistance is only slightly greater than the horizontal thrust that tends to cause sliding. For a gravity dam on an earth foundation, the factor of safety should be at least 2 and preferably 3.

EXAMPLE.—If the dam in the example of the preceding article rests on a foundation bed of solid rock and the coefficient of friction between the dam and the rock may be taken as .75, what is the factor of safety against sliding for the trial section under consideration?

SOLUTION.—In this case, the horizontal component of the resultant force acting on the dam is the sum of the horizontal component of the water pressure and the ice pressure, or 657,000+36,000=693,000 lb. Also, the vertical component of the resultant force, as previously determined, is 1,110,000 lb., and the frictional resistance is 1,110,000×.75=833,000 lb. Hence, the factor of safety against sliding is 833,000÷693,000=1.20. Ans.

35. Sliding on Horizontal Plane Above Base.—The horizontal forces acting on the dam above any horizontal plane tend to cause the upper part of the dam to slide on that plane. In this case sliding is resisted by the shearing strength of the masonry and by the friction between the upper part of the dam and the lower part. Care is usually taken in constructing a concrete dam to avoid a horizontal joint at the end of a day's work and to provide suitable keys for bonding new work to old. Also, when dams are constructed of stone masonry, rubble work is considered preferable to cut-stone work because of the better bond. Hence, the shearing strength of the masonry in a dam is generally considerable. However, if it is disregarded, the investigation of the dam for sliding at any plane above the base is similar to that for sliding on the foundation, the forces to be considered being the weight of the portion of the dam above the plane and the water pressure on that part of the dam. Also, the coefficient of friction is that for the material of the dam on itself. Owing to the fact that the shearing strength of the masonry provides a sufficient factor of safety, the frictional resistance at the joint need be but slightly greater than the force tending to cause sliding.

EXAMPLES FOR PRACTICE

1. The back of a dam has a slope of 2 inches horizontal to 1 foot vertical. If there is a deposit of earth 18 feet deep behind the dam, and its effective weight is taken as 60 pounds per cubic foot and its angle of repose as 25°, determine (*a*) the horizontal component and (*b*) the vertical component of the pressure exerted on the dam by the deposited material. Ans. $\begin{cases} (a)\ 3{,}940 \text{ lb.} \\ (b)\ 1{,}620 \text{ lb.} \end{cases}$

2. For the cross-section shown in Fig. 11 and the conditions given in the example of Art. 33, find the distance from the down-stream face of the dam to the point where the resultant of all the forces acting above the horizontal section through *b* cuts that section. Ans. 21.17 ft.

3. What is the factor of safety against sliding at the section considered in the preceding example, if the coefficient of friction is assumed to be .7? Ans. 1.23

CRUSHING

36. Crushing at Face.—In the case of a very high dam, great pressure is concentrated on the foundation and the masonry at the face. If the foundation is of good rock, it can safely withstand any stress to which the structure may be subjected, but the masonry of the dam may be crushed unless the maximum unit pressure is limited by making the dam sufficiently wide. Since the masonry constructed under the conditions prevailing at the bottom of a large dam does not have the strength indicated by tests on laboratory samples, and since the crushing at the toe of the dam is liable to lead to overturning or sliding of the structure, it is generally considered best to provide an ample margin of safety. In the early concrete dams that were built, the allowable compressive unit stress was taken as about 16 tons per square foot. However, this stress has been gradually increased and, in the design of many gravity dams, has been taken as 30 tons per square foot and even higher.

The vertical unit pressure calculated by formula 1, Art. 30, is not the maximum stress in the masonry. To the value thus calculated must be added the unit pressure due to uplift, if there is any at the face of the dam; and the sum is the total vertical pressure. Moreover, the maximum unit pressure is produced in an inclined direction. Various methods have been proposed for determining the inclined stresses in dams, but all these methods introduce an element of approximation. An assumption commonly made for usual conditions is that the greatest unit pressure is produced on a section perpendicular to the face of the dam and that the amount of that unit pressure is

$$p' = p_m \sec^2 Y$$

in which p' = maximum inclined unit stress at face of dam;
p_m = maximum vertical unit pressure;
Y = angle that face of dam makes with the vertical.

In the case of comparatively low dams, it is sufficient to determine the maximum vertical compressive unit stress and to compare it with a permissible value that is assumed low

enough to allow for the fact that the inclined unit stresses are considerably higher. However, for high dams, the inclined compressive unit stresses should be carefully investigated.

37. Crushing at Back.—When the reservoir behind the dam is empty, the greatest stress in the masonry occurs at the back. For this condition, the only force considered is the weight of the dam itself. If the line of action of this weight should cut the base of the dam outside of the middle third, there would be a tendency for the dam to lift up at the face. Although it is hardly probable that the dam would overturn backwards under these circumstances, it is customary to design the dam so that its weight will act within the middle third of each section. The formulas in Art. 30 apply also to this case, but the pressures at the face and back are interchanged. Thus, when the line of action of the weight cuts the base within the middle third, the pressure p_1 at the back is

$$p_1=\frac{R_v}{b}\left(1+\frac{6e}{b}\right) \qquad (1)$$

In this case, no additional pressure due to uplift is acting. Also, the maximum unit stress at the back may be assumed as

$$p_1'=p_1 \sec^2 Y' \qquad (2)$$

in which p_1' = maximum unit stress at back;
p_1 = maximum vertical unit stress;
Y' = angle that back of dam makes with the vertical.

DESIGN OF GRAVITY DAMS

GENERAL CONSIDERATIONS

38. Nature of Problem.—In the design of a gravity dam of moderate height, the only feature that usually has to be considered is the position of the resultant force with respect to the middle third of the section at any depth and at the base. Sliding and crushing become important only when the dam reaches a considerable height. Nevertheless, the economical shape of the cross-section of a dam of medium height is such that no simple equation can be derived for determining the required width at a given depth. The customary procedure is to assume widths at certain levels in the cross-section in accordance with past experience with similar dams, to investigate the stability of the part of the dam above each such level, and to alter the widths as required.

39. Top Width.—It is usually found economical to make a dam fairly wide at the top, even where the top of the dam is not to be used as a roadway and there is no likelihood of ice exerting a pressure on the back of the dam. When economy is the only consideration, the top width is made a certain percentage of the height of the dam, varying from about 10 per cent. of the maximum height for high dams to 15 per cent. for low dams. However, the required width of the roadway and the effect of ice pressure at a comparatively high water level may be determining factors in establishing the top width.

40. Superelevation.—In order to provide for unforeseen floods, ice jams, or wave action, the top of a dam is usually placed above the normal high-water level, the superelevation or freeboard being, in general, about 5 per cent. of the height of the dam. However, there are practical limitations, and the superelevation is rarely less than 2 feet or more than 10 feet. The design of the spillway and the character of local flood flows influence the superelevation to some extent. A formula for computing the probable height of waves, which is given by Stevenson, is

$$h_w=1.5\sqrt{F}-\sqrt[4]{F}+2.5$$

in which h_w = height of wave, in feet;
F = fetch, or range of open water exposed to sweep of wind, in miles.

However, a masonry dam would not usually be damaged by waves washing over the crest of the dam.

41. Division of Dam into Zones.—In designing a cross-section for a dam, the required width at different levels is determined by different conditions of stability. As shown in Fig. 11, the top portion of the dam usually has a vertical back and face. Where there is ice pressure at a relatively high water

level and the superelevation has been established, the top width must be sufficient to prevent sliding at the level of the ice. After the top width has been established, the next step is to locate the point f at such a depth that the resultant of the forces acting on the dam above the horizontal section through f will cut that section at the down-stream edge of its middle third. Between the ice level and the level at the point f, the thickness of the masonry is ample, because the resultant of the forces above any section between these levels must cut that section within the middle third, and the horizontal thrust exerted by the water on the back of the dam above the section is much less than the resistance to sliding provided by the weight of the masonry between the ice level and the section.

The requirement that the resultant should pass through the down-stream edge of the middle third of each section governs the width of the dam until the level j is reached, where the line of action of the weight of the dam passes just inside the up-stream edge of the middle third. For the remainder of a dam of moderate height, the width need only be such that for maximum loading the resultant force passes through the down-stream edge of the middle third and for an empty reservoir the line of action of the weight of the dam passes through the up-stream edge of the middle third. In high dams, other stages may be reached. In such dams the consideration that determines the width in the next stage is usually the maximum unit pressure at the face; for a gravity dam, the inclined unit stresses are always greater at the face than at the back. Of course, sliding must be considered in every stage, although ordinarily the required width is governed by other considerations. However, for the lower stages of high dams in which the allowable crushing unit stress is great, sliding may determine the required width. Each portion of the height of the dam, in which a different condition or combination of conditions governs the width, may be termed a zone.

GENERAL METHOD OF DESIGN

42. Outline of Procedure.—Owing to the fact that the up-stream and down-stream faces of a dam have different slopes at various heights, it is neither sufficient nor economical to assume a tentative cross-section for the entire height and to judge the stability of the entire structure by investigating only the base. Instead, the dam is imagined to be divided into a series of horizontal layers and the required width at the bottom of each layer is determined. The top layer is designed first and then each lower layer is considered in turn. Hence, in establishing the width at the bottom of any layer, the dimensions of the layers above the one in question may be considered fixed, and only the required width of that layer is altered when it is necessary to assume a new trial section.

43. Determination of Top Width to Resist Ice Pressure. Where ice pressure may be neglected, as is often the case, the top width of the dam is established either as a percentage of the height of the dam or by the required width for a roadway. However, where ice pressure is likely to be quite high and an allowance for it must be made, the width required to resist that pressure may be found as follows: Let P_i represent the total ice pressure per linear foot of dam; h_i, the vertical distance from the bottom of the ice to the top of the dam, in feet; t, the required top width, in feet; w_m, the weight of the masonry, in pounds per cubic foot; and f, the coefficient of friction for the masonry on itself. Then the force that tends to cause sliding is P_i and, since the volume of masonry that resists sliding is $h_i t$, the weight of the masonry is $h_i t w_m$. Hence, for stability,

$$P_i = f h_i t w_m$$

or

$$t = \frac{P_i}{f h_i w_m}$$

EXAMPLE.—A concrete dam is to resist an ice pressure of 40,000 pounds per linear foot at a section 15 feet below its top. If the coefficient of friction for the masonry is taken as .7, what is the required top width of the dam?

SOLUTION.—Here, $P_i = 40{,}000$ lb., $f = .7$, $h_i = 15$ ft., and $w_m = 150$ lb. per cu. ft. Then,

$$t = \frac{P_i}{f h_i w_m} = \frac{40{,}000}{.7 \times 15 \times 150} = 25.4 \text{ ft. Ans}$$

44. Level at Which Down-Stream Face Begins to Slope. After the superelevation and top width have been selected, the

next step is to locate the point f, Fig. 11, at which the downstream face of the dam begins to slope. In locating this point, two conditions of loading may have to be considered: (1) water at its highest level and no ice pressure; (2) water at a lower level, with ice pressure. For either condition, the horizontal distance from f to the line of action of the resultant of the forces acting on the part of the dam above that point should be one-third of the top width. Since no simple formula can be derived for locating f directly, the procedure is to assume a value for the vertical distance from the top of the dam to f

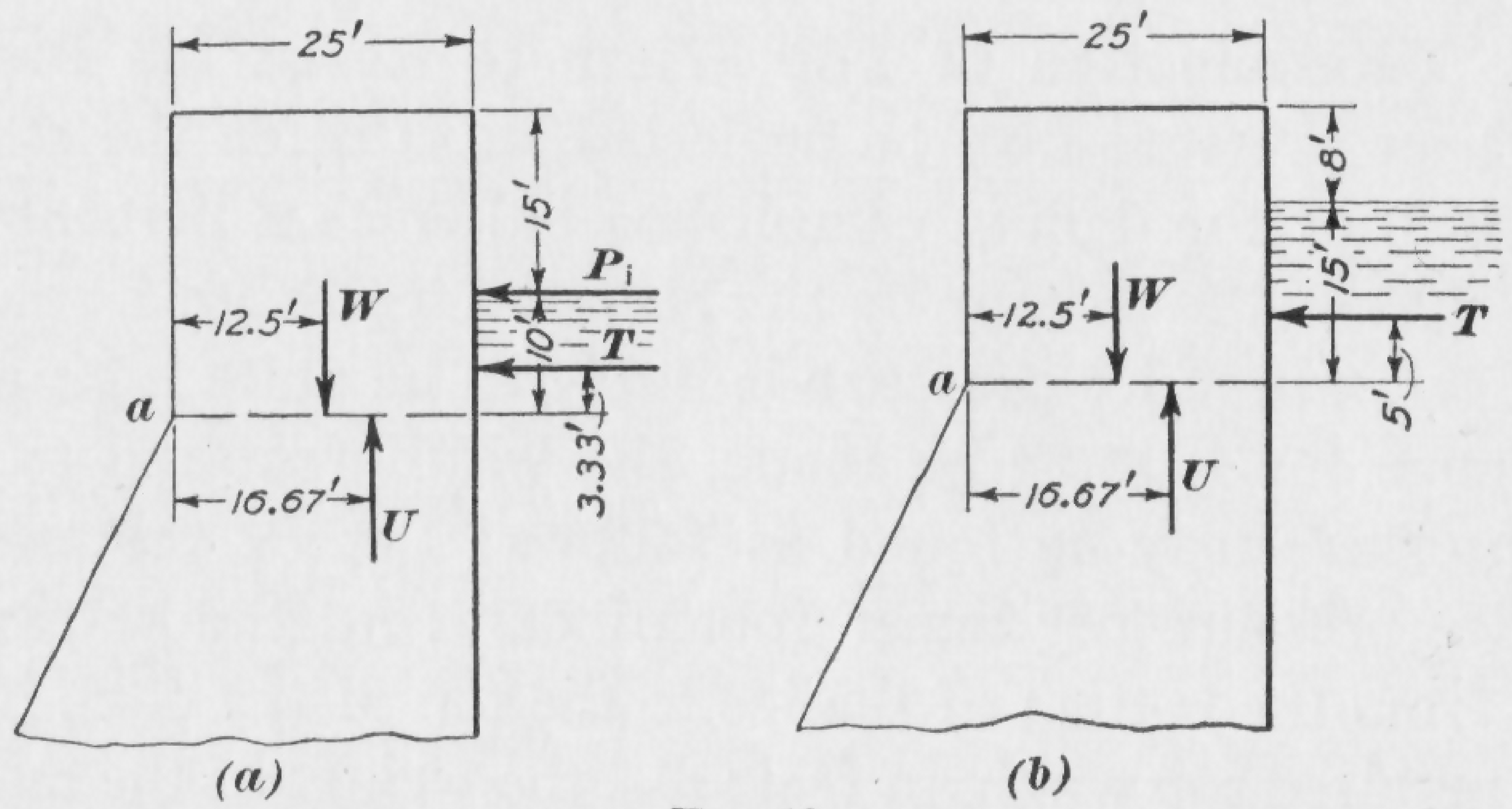

FIG. 12

and to investigate the stability of the dam at that level. As a guide in selecting a trial value when ice pressure is neglected, the depth from the high-water level to f may be taken as 1.5 times the top width. For the maximum ice pressure, the depth from the ice level to f may be taken as .4 times the top width. When ice pressure is considered, the location of the point f is generally governed by the loading in which that pressure is included. If it is found that the resultant force cuts the section outside of the middle third, the correct position of the point of slope is above the assumed location. In case the resultant cuts the section too far within the middle third, say, more than 2 inches from the edge, the point of slope may be taken below the assumed position.

EXAMPLE.—If the top width of the dam in the example of the preceding article is made 25 feet and the superelevation above the normal high-water level is 8 feet, what should be the vertical distance from the top of the dam to the point where the face starts to slope?

SOLUTION.—As specified in the preceding article, the distance from the top of the dam to the level at which the ice pressure is assumed to act is 15 ft. Here, the location of the point of slope is probably determined by the loading when there is ice, and the distance from the ice level to the point is taken as $.4 \times 25 = 10$ ft. Then the distance from the top of the dam to that point is $15+10=25$ ft. For the condition of loading under investigation, the forces to be considered are the weight of the masonry above the point of slope, the water pressure on the back of the dam above that point, the uplift pressure on the horizontal section at that point, and the ice pressure. The part of the dam under consideration and the forces acting on it are shown in Fig. 12 (a). These forces and the moment of each about the point of slope a may be tabulated as follows:

Force	Amount	Arm	Moment About a
W	$25\times25\times150 = 93{,}750$	12.5	1,172,000
T	$31.25\times10^2 = 3{,}130$	3.33	10,000
U	$31.25\times.5\times25\times10 = 3{,}910$	16.67	65,000
P_i	40,000	10	400,000

The vertical component of the resultant force is $W-U=93{,}750-3{,}910=89{,}840$ lb., and the resultant moment about a is $1{,}172{,}000-10{,}000-65{,}000-400{,}000=697{,}000$ ft.-lb. Hence, the horizontal distance from a to the resultant force is $697{,}000 \div 89{,}840 = 7.76$ ft. Since this is less than $\frac{1}{3}\times25=8.33$ ft., the resultant passes outside of the middle third of the section. Hence, the point of slope should be above the assumed position.

If the distance from the top of the dam is assumed to be 23 ft., the forces and their moments are:

Force	Amount	Arm	Moment About a
W	$25\times23\times150 = 86{,}250$	12.5	1,078,000
T	$31.25\times8^2 = 2{,}000$	2.67	5,000
U	$31.25\times.5\times25\times8 = 3{,}130$	16.67	52,000
P_i	40,000	8	320,000

In this case, the vertical component of the resultant is $86{,}250-3{,}130=83{,}120$ lb., the resultant moment is $1{,}078{,}000-5{,}000-52{,}000-320{,}000=701{,}000$ ft.-lb., and the distance from a to the resultant is $701{,}000\div83{,}120=8.43$ ft. Since the resultant passes only about $1\frac{1}{4}$ in. inside of the middle third, the point of slope may be located 23 ft. below the top of the dam.

For the condition of loading with the high-water level 8 ft. below the top of the dam and no ice pressure, the forces are as shown in Fig. 12 (b). In this case, the amounts of the forces and their moments are as follows:

Force	Amount	Arm	Moment About a
W	$25\times23\times150 = 86{,}250$	12.5	1,078,000
T	$31.25\times15^2 = 7{,}030$	5	35,000
U	$31.25\times.5\times25\times15 = 5{,}860$	16.67	98,000

The vertical component of the resultant is $86{,}250-5{,}860=80{,}390$ lb., the resultant moment is $1{,}078{,}000-35{,}000-98{,}000=945{,}000$ ft.-lb., and the distance from a to the resultant is $945{,}000\div80{,}390=11.76$ ft. Hence, the resultant lies well within the middle third of the section and the distance from the top of the dam to the point of slope is made 23 ft. Ans.

45. Some designers assume the high-water level to be at the top of the dam. The point at which the down-stream face of the dam begins to slope may then be determined by direct calculation. Also, in the case of dams for which ice pressure may be disregarded and the top width is relatively small, this assumption often produces a dam profile of somewhat better appearance than that obtained by assuming the high-water level at a certain distance below the top of the dam.

EXAMPLE.—If the high-water level is assumed to be at the top of the dam, the top width is 10 feet, and the dam is to resist only water pressure and uplift, what is the distance from the top of the dam to the point where the face of the dam should begin to slope?

SOLUTION.—If h represents the distance from the top of the dam to the required point, the horizontal component of the water pressure above the level of that point is

$$T=31.25\,h^2$$

The weight of the concrete above the point of slope is

$$W=150\times10\,h=1{,}500\,h$$

and the uplift for a head h and a width of 10 ft. is

$$U=31.25\times.5\times10\,h=156.25\,h$$

The resultant of the forces T, W, and U will pass through the middle-third point of the section that is nearer to the down-stream face of the dam when the algebraic sum of their moments about that point is zero. As the arms of the forces T, W, and U about that point are, respectively, $\frac{1}{3}\,h$, $\frac{1}{6}\times10$, and $\frac{1}{3}\times10$, the equation for the algebraic sum of the moments is

$$-31.25\,h^2\times\tfrac{1}{3}\,h+1{,}500\,h\times\tfrac{1}{6}\times10-156.25\,h\times\tfrac{1}{3}\times10=0$$

Whence, $10.42\,h^3=2{,}500\,h-521\,h=1{,}979\,h$

and $h=13.78$ ft. Ans.

46. Selection of Lower Sections for Investigation.—Below the point at which the face of the dam starts to slope, the choice of the sections at which the width of the dam is calculated is entirely a matter of judgment. A convenient rule is to make the vertical distance between two consecutive sections about 15 per cent. of the distance from the top of the dam to the upper section. However, some designers make the minimum distance between sections about 10 feet. No attempt is made to locate exactly the sections of division between the various zones. After the points on the up-stream and down-stream surfaces are determined at the selected sections, they are usually connected by smooth curves.

47. Width in Zone With Vertical Back and Sloping Face. In the zone in which the line of action of the weight passes within the middle third without necessitating the battering of the back of the dam, as between f and j in Fig. 11, the procedure consists in selecting a convenient depth, assuming the required width at the section, and investigating the stability of the part of the dam above that section. If the resultant force cuts the section outside of the middle third, the width must be increased; on the other hand, if the resultant passes too far within the middle third, it is economical to use a smaller width. Since the back of the dam is vertical, it is customary in locating the resultant to take moments about a point on the back of the dam.

When no other information is available, it may be assumed that the batter of the face of the dam in this zone is 1 horizontal to 2 vertical, or that the increase in width between two sections is equal to half of the vertical distance between the sections. Also, in case the trial width at a section is found unsatisfactory, it is useful in selecting the new width for investigation to remember that, in general, the position of the resultant force is moved about 1 inch for a change of 1 foot in the width of section.

EXAMPLE.—What is the required width of a section 60 feet below the top of the dam considered in Arts. 43 and 44? It may be assumed that the dam will be stable against sliding and crushing, and that the slope of the down-stream face is uniform to the section in question.

SOLUTION.—The vertical distance from the point at which the face starts to slope to the section under investigation is 60−23=37 ft., and the increase in width from that point of slope is assumed to be $\frac{1}{2}\times 37=18.5$ ft. Since the top width is 25 ft., the width at the section under consideration is assumed to be 25+18.5=43.5 ft. The part of the dam above the section, with the forces acting on it for the condition when ice pressure is included, is shown in Fig. 13 (*a*). In this case, the weight of the masonry above the section is divided into two parts, namely,

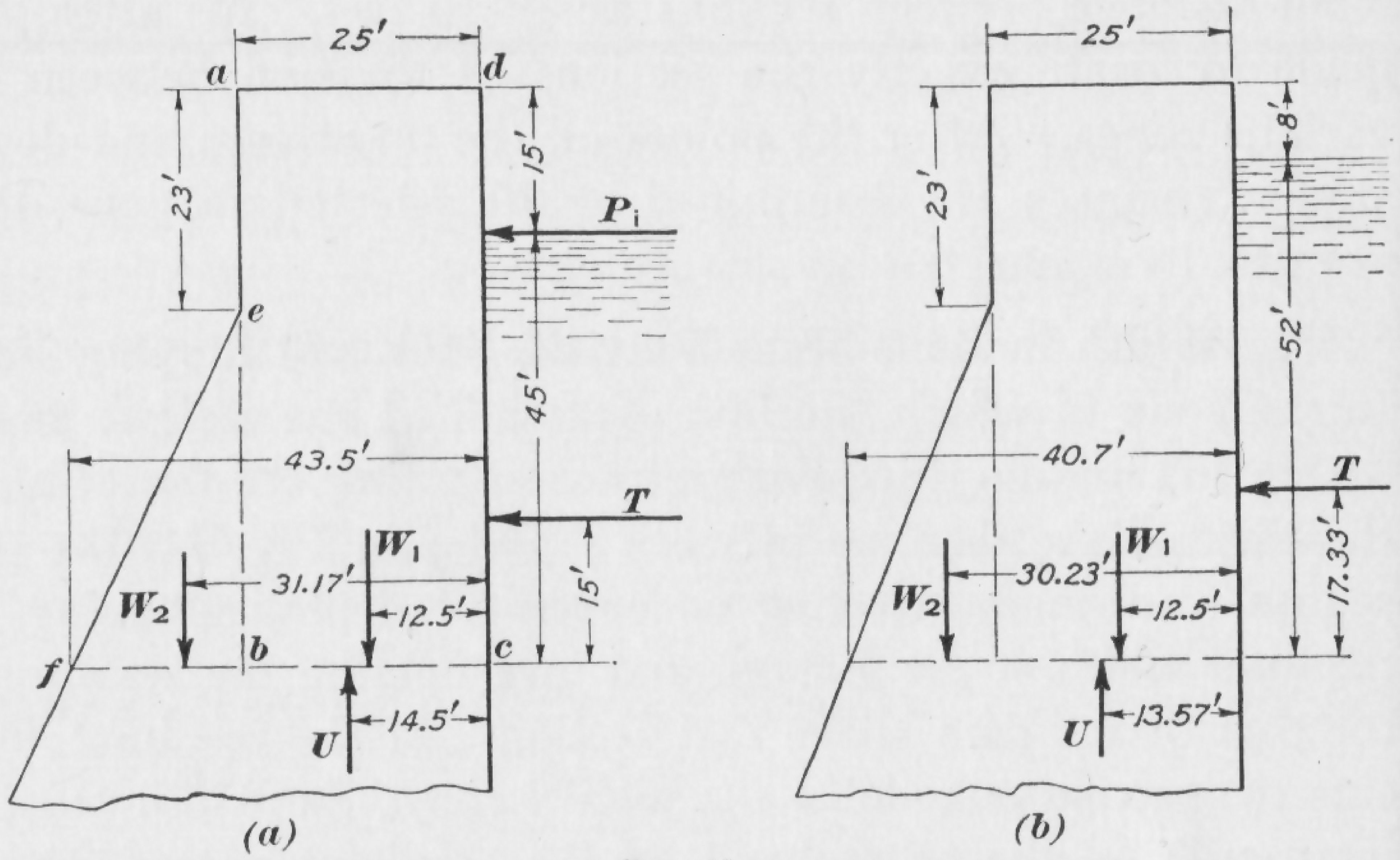

FIG. 13

the weight W_1 of the rectangular portion *abcd* and the weight W_2 of the triangular portion *bef*. The forces and their moments about the point *c* on the back of the dam may be tabulated as follows:

Force	Amount	Arm	Moment About *c*
W_1	25×60×150 =225,000	12.5	2,813,000
W_2	$\frac{1}{2}$×18.5×37×150 = 51,300	31.17	1,599,000
T	31.25×45² = 63,300	15	950,000
U	31.25×.5×43.5×45= 30,600	14.5	444,000
P_i	40,000	45	1,800,000

The vertical component of the resultant force is $W_1+W_2-U=225{,}000+51{,}300-30{,}600=245{,}700$ lb., the resultant moment is 2,813,000+1,599,000+950,000−444,000+1,800,000=6,718,000 ft.-lb., and the distance from *c* to the resultant is 6,718,000÷245,700=27.34 ft. The distance from *c* to the down-stream edge of the middle third is $\frac{2}{3}\times 43.5=29$ ft. Hence, the width can be reduced somewhat.

If the position of the resultant remained unchanged, the distance 27.34 ft. would be two-thirds of the required width. In other words, that width would be $27.34\div\frac{2}{3}=41.01$, or say 41 ft. However, since the

difference between this value and the previous trial width of 43.5 ft. is 2.5 ft., the resultant will probably move about $2\frac{1}{2}$ in., or say .2 ft. toward the point *c*. Hence, the distance from *c* to the resultant for the new trial width may be assumed to be 27.34−.2=27.14 ft. and that width may be taken as $27.14\div\frac{2}{3}=40.71$, or say 40.7, ft. For this dimension, $W_2=\frac{1}{2}\times 15.7\times 37\times 150=43{,}600$ lb., $U=31.25\times .5\times 40.7\times 45=28{,}600$ lb., and the moments of these forces are, respectively, 43,600×30.23=1,318,000 ft.-lb. and 28,600×13.57=388,000 ft.-lb. The other values are the same as for a width of 43.5 ft. Therefore, $W_1+W_2-U=225{,}000+43{,}600-28{,}600=240{,}000$ lb., and the resultant moment is 2,813,000+1,318,000+950,000−388,000+1,800,000=6,493,000 ft.-lb. The distance from *c* to the resultant for the width of 40.7 ft. is then 6,493,000÷240,000=27.05 ft., which is only 1 in. less than $\frac{2}{3}\times 40.7=27.13$ ft.

When the condition of loading for high-water level without ice pressure is considered, the forces acting on the dam are as shown in Fig. 13 (*b*). As found by investigation, the width of 40.7 ft. is ample for this condition. Also, it is evident that the resultant of the weights W_1 and W_2 lies well within the middle third of the section. Therefore, the required width is 40.7 ft. Ans.

48. Width in Zone With Battered Back.—When the back of the dam must be battered in order that the line of action of the weight will lie inside the middle third of the section, it is necessary to determine by a series of trials not only the width of the section but also the amount of the projection at the back. The assumptions of the first trial width and projection are based entirely on experience and judgment. In locating the resultant for each trial section, it is convenient to take the reference point some distance outside the dam, because the exact position of the back of the dam is not known beforehand. After the positions of the resultant forces have been determined for the assumed section, both for the weight of the masonry alone and for the maximum loading, the calculated distances from the reference point to the two resultants are compared with the distances from the reference point to the edges of the middle third of the assumed section.

There are four possible conditions, and these are shown in Fig. 14, where *a* is the reference point, *bc* is the section of the dam under consideration, *W* is the resultant weight of the masonry, and *R* is the resultant force for maximum loading. In view (*a*), both *W* and *R* cut the section within the middle

third; in order to make the resultants pass nearer the edges of the middle third, both the width of the section and the projection at the back should be reduced. In view (*b*), both *W* and *R* pass outside the middle third and, therefore, both the width and the projection should be increased. For either of these two conditions, new values of the width and the projection to serve

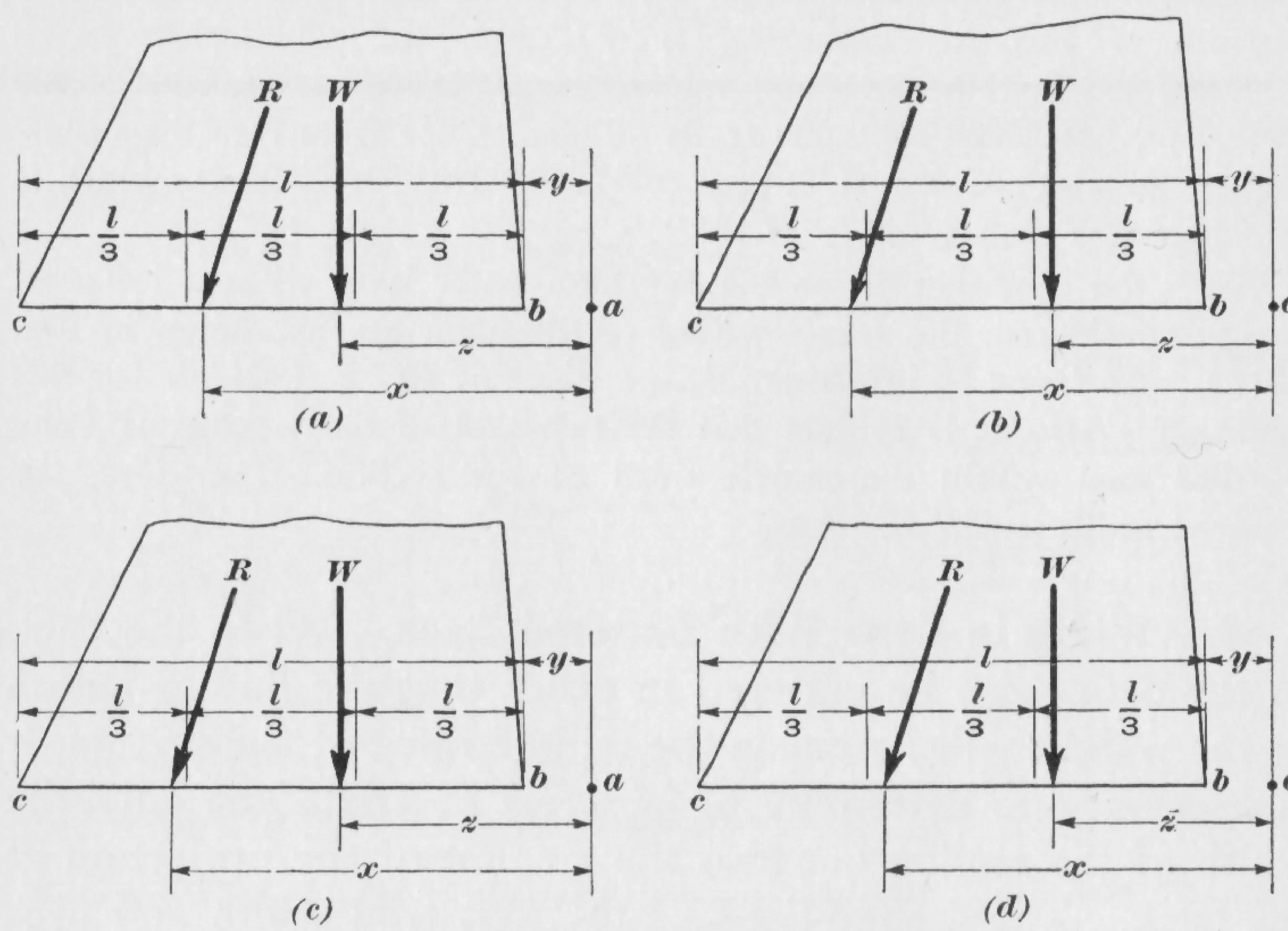

FIG. 14

as guides in selecting the next trial values are found by applying the following formulas:

$$l=3(x-z) \qquad (1)$$

$$y=z-\tfrac{1}{3}\,l \qquad (2)$$

in which l = guide value of width of dam;
x = computed distance from reference point to resultant force under maximum loading;
z = computed distance from reference point to line of action of weight of masonry;
y = guide value of distance from reference point to back of dam.

The next trial values of l and y are taken somewhere between the values first assumed for calculating x and z and those found

by means of formulas 1 and 2; the positions of the resultants for the new section are then located; and the values of l and y are again computed. Additional trials are made until the value of x for the last trial section is either equal to or slightly less than the distance $y+\frac{2}{3}\,l$ from the reference point to the down-stream edge of the middle third of the section, and the computed value of z is either equal to or slightly greater than $y+\frac{1}{3}\,l$.

In Fig. 14 (*c*), the resultant *W* passes inside the middle third and the resultant *R* passes outside the middle third; and in view (*d*) the resultant *W* is outside and the resultant *R* inside the middle third. For either of these conditions, the preceding formulas for guide values of l and y do not usually give satisfactory results, and no similar formulas can be derived for the same purpose. Hence, the next trial values of y and l must be determined from a study of the positions of the resultants with respect to the edges of the middle third. In assuming these values, the following principles are helpful. When the section is altered, the locations of the edges of the middle third and also the positions of the resultants are changed. If y is changed and l is kept constant, the edges of the middle third are moved the full amount of the change in y, whereas the resultants move much less. Also, if y is kept constant and l is changed, the up-stream edge of the middle third moves through a distance equal to one-third of the change in l and the down-stream edge of the middle third moves two-thirds of the change in l, whereas the resultants move about 1 inch per foot of change in l.

Since a change in l produces a comparatively small change in the positions of the resultants and the up-stream edge of the middle third, a good trial value of y can be determined as follows: The distance from the up-stream edge of the middle third of the assumed section to the line of action of the weight alone is first found. Then the back of the dam is moved a distance slightly greater than this so that the resultant weight will pass nearer the edge of the middle third. If the resultant weight passes too far inside the middle third, the distance y from the reference point to the back of the dam is increased; on the other hand, if the resultant weight lies outside the middle third, the distance y is reduced. The new trial width l is selected mainly

by judgment; the foregoing principles being applied in making a rough estimate of the required change in l

EXAMPLE.—Assume that in the tentative cross-section shown in Fig. 11 the positions of the points b and j are satisfactory. As stated in the example in Art. 33, the ice pressure is 36,000 pounds per linear foot and it acts at a distance of 15 feet below the top of the dam; also, the uplift pressure at the down-stream face is zero. For this loading determine the required width of base and projection at the back so that the resultants will pass just within the middle third.

SOLUTION.—The areas of the portions of the dam whose weights are W_1, W_2, W_3, W_4, and W_5, and the amounts of the forces U, T, and V, were calculated in the example of Art. 33. For convenience in solving this problem, the reference point m about which moments are taken is assumed to be 10 ft. to the right of i. Then, the forces, their arms with respect to m, and their moments may be tabulated as follows:

Force	Amount	Arm	Moment About m
W_1	1,260×150=189,000	86	16,254,000
W_2	2,520×150=378,000	51	19,278,000
W_3	1,580×150=237,000	44	10,428,000
W_4	3,200×150=480,000	20	9,600,000
W_5	240×150= 36,000	8	288,000
V	39,400	7	276,000
U	249,000	40.67	10,127,000
P_i	36,000	145	5,220,000
T	657,000	48.33	31,753,000

The vertical component of the resultant for maximum load is equal to $W_1+W_2+W_3+W_4+W_5+V-U=1{,}110{,}400$ lb.; the resultant moment, which is found by adding the moments of all the forces except U and subtracting the moment of U from the sum, is 82,970,000 ft.-lb.; and the distance from the reference point m to the resultant for maximum load is

$$x=82{,}970{,}000\div1{,}110{,}400=74.72 \text{ ft.}$$

Also, for the weight of the masonry alone, the resultant moment is the sum of the moments of W_1, W_2, W_3, W_4, and W_5, or 55,848,000 ft.-lb. Since the weight of the masonry is 1,320,000 lb., the distance from m to the line of action of the total weight is

$$z=55{,}848{,}000\div1{,}320{,}000=42.31 \text{ ft.}$$

The distance from the reference point to the back of the dam is $y=10-6=4$ ft., to the up-stream edge of the middle third is $y+\frac{1}{3}l=4+\frac{1}{3}\times110=40.67$ ft., and to the down-stream edge of the middle third is $y+\frac{2}{3}l=4+\frac{2}{3}\times110=77.33$ ft. In this case, x is considerably less than $y+\frac{2}{3}l$ and therefore the resultant for maximum loading passes well inside the middle third; also, z is considerably greater than $y+\frac{1}{3}l$ and the line of action of the weight lies well inside the middle third.

Since both resultants are inside the middle third, formulas 1 and 2 are applied. Thus,

$$l=3(x-z)=3\times(74.72-42.31)=97.23 \text{ ft.}$$

$$y=z-\tfrac{1}{3}l=42.31-\tfrac{1}{3}\times97.23=9.9 \text{ ft.}$$

Hence, for the new trial section, l will be taken as 104 ft. and y as 7.5 ft. For these dimensions the distance mk, Fig. 11, becomes 7.5 ft. and ki is $10-7.5=2.5$ ft.; also, $ka=104$ ft. and $ca=104-2.5-20-42=39.5$ ft. The forces and their moments are then as follows:

Force	Amount	Arm	Moment
W_1	$\frac{1}{2}\times39.5\times60\times150$ =177,800	85.17	15,143,000
W_2	378,000	51	19,278,000
W_3	237,000	44	10,428,000
W_4	480,000	20	9,600,000
W_5	$\frac{1}{2}\times2.5\times80\times150$ = 15,000	9.17	138,000
V	$2.5\times\left(\frac{65+145}{2}\right)\times62.5$= 16,400	8.75	144,000
U	31.25×.5×104×145 =235,600	42.17	9,935,000
P_i	36,000	145	5,220,000
T	657,000	48.33	31,753,000

Then, for the weight of the dam alone the vertical component of the resultant is $W_1+W_2+W_3+W_4+W_5=1{,}287{,}800$ lb. and the resultant moment is the sum of the moments of these weights, or 54,587,000 ft.-lb. Also, for maximum load, the vertical component is $1{,}287{,}800+16{,}400-235{,}600=1{,}068{,}600$ lb., and the moment is $54{,}587{,}000+144{,}000+5{,}220{,}000+31{,}753{,}000-9{,}935{,}000=81{,}769{,}000$ ft.-lb. Hence,

$$x=81{,}769{,}000\div1{,}068{,}600=76.52 \text{ ft.}$$

and

$$z=54{,}587{,}000\div1{,}287{,}800=42.39 \text{ ft.}$$

Since $y+\frac{1}{3}l=7.5+\frac{1}{3}\times104=42.17$ ft. and $y+\frac{2}{3}l=7.5+\frac{2}{3}\times104=76.83$ ft., both resultants still lie within the middle third, but are much closer to the edges. More accurate values of y and l may be found by again applying formulas 1 and 2. Thus,

$$l=3(x-z)=3\times(76.52-42.39)=102.39 \text{ ft.}$$

and

$$y=z-\tfrac{1}{3}l=42.39-\tfrac{1}{3}\times102.39=8.26 \text{ ft.}$$

If the new trial values of l and y are taken as 103.5 ft. and 7.9 ft., respectively, then mk, Fig. 11, becomes 7.9 ft., ca is 39.4 ft., and the results are as follows:

Force	Amount		Arm	Moment
W_1	$\frac{1}{2}\times 39.4\times 60\times 150$	$=177{,}300$	85.13	15,094,000
W_2		378,000	51	19,278,000
W_3		237,000	44	10,428,000
W_4		480,000	20	9,600,000
W_5	$\frac{1}{2}\times 2.1\times 80\times 150$	$=12{,}600$	9.3	117,000
V	$2.1\times\left(\frac{65+145}{2}\right)\times 62.5=$	13,800	8.95	124,000
U	$31.25\times .5\times 103.5\times 145$	$=234{,}500$	42.4	9,943,000
P_i		36,000	145	5,220,000
T		657,000	48.33	31,753,000

Then, $$x=\frac{81{,}671{,}000}{1{,}064{,}200}=76.74 \text{ ft.}$$

and $$z=\frac{54{,}517{,}000}{1{,}284{,}900}=42.43 \text{ ft.}$$

Also, $$y+\tfrac{1}{3}l=7.9+\tfrac{1}{3}\times 103.5=42.4 \text{ ft.}$$

and $$y+\tfrac{2}{3}l=7.9+\tfrac{2}{3}\times 103.5=76.9 \text{ ft.}$$

Since both resultants are only slightly inside the edges of the middle third, the width of 103.5 ft. and the projection at the back of $10-7.9=2.1$ ft. may be considered satisfactory. Ans.

49. Zone in Which Compressive Stresses Govern Width. In the design of a high dam, when the zone is reached in which the compressive unit stresses govern the width, the first step in the calculations for any level is to determine the amounts and positions of the resultant forces for full reservoir and for empty reservoir. Both these resultants should pass well within the middle third of the section. The next step is to compute the maximum inclined unit pressure at the face. In case the actual stress exceeds the allowable value, it should preferably be reduced by decreasing the angle Y that the face of the dam makes with the vertical. This may be accomplished by increasing the width of the dam at higher levels. However, the unit stress may also be lowered by increasing the width of the section under consideration.

EXAMPLE.—What is the maximum unit stress at the toe of the dam in the example of the preceding article?

SOLUTION.—In formula 1, Art. 30, $R_v=1{,}064{,}200$ lb. and $b=103.5$ ft., as determined in the preceding article. Also, the distance from the back of the dam to the center of the base is $\frac{1}{2}\times 103.5=51.75$ ft. and to the resultant force is $76.74-7.9=68.84$ ft. Hence, the eccentricity is $e=68.84-51.75=17.09$ ft., and the vertical upward reaction of the foundation at the toe is

$$p=\frac{R_v}{b}\left(1+\frac{6e}{b}\right)=\frac{1{,}064{,}200}{103.5}\times\left(1+\frac{6\times 17.09}{103.5}\right)=20{,}500 \text{ lb. per sq. ft.}$$

Since there is no uplift at the toe in this case, the maximum vertical unit pressure is 20,500 lb. per sq. ft. At the bottom of the dam, the slope of the face is 39.4 ft. horizontally in 60 ft. vertically and $\tan Y=\frac{39.4}{60}=.657$. Therefore, $Y=33°\ 18'$ and, by the formula of Art. 36, the maximum inclined unit stress is

$$p'=p_m \sec^2 Y=20{,}500 \sec^2 33°\ 18'=29{,}400 \text{ lb. per sq. ft.} \quad \text{Ans.}$$

EXAMPLES FOR PRACTICE

1. A gravity dam is to have a top width of 18 feet and a superelevation of 5 feet. If no allowance need be made for ice pressure, determine the vertical distance from the top of the dam to the point where the face starts to slope. Ans. 32.5. ft.

2. Find the required width of the dam in the preceding example at a depth of 60 feet below the top. It may be assumed that the downstream face will have a uniform slope to the section under investigation and that there will be sufficient resistance to sliding and crushing. Ans. 32.7 ft.

3. If the batter of the back of the dam in the preceding examples starts at the depth of 60 feet, find (*a*) the required width and (*b*) the projection at the back at a depth of 72 feet. Ans. (*a*) 42.1 ft. (*b*) 1.0 ft.

4. What is the maximum compressive unit stress in the dam at the section in example 3? Ans. 15,530 lb. per sq. ft.

TYPICAL DESIGN OF NON-OVERFLOW SECTION OF GRAVITY DAM

50. The method of procedure in designing the cross-section of a gravity dam where the water level does not rise above the top of the dam is illustrated in the following example:

EXAMPLE.—It is required to design a plain-concrete dam whose total height is 110 feet. The superelevation is taken as 10 feet and, since no allowance is made for ice pressure, the top width is also made 10 feet. The coefficient of friction for the masonry on itself and for the dam on its

foundation is .75. Tail-water pressure on the face of the dam is neglected. The weight of the concrete is assumed as 150 pounds per cubic foot and the allowable compressive unit stress at the face of the dam is 40,000 pounds per square foot. In accordance with the usual practice, the value of the coefficient k in the formula for uplift is taken as .5.

Solution.—*Location of Top of Slope on Face.*—In this case, there is no ice pressure and, as a trial, the depth from the water level to the top of the slope on the face will be taken as 1.5 times the top width or $1.5\times10=15$ ft. Hence, the distance from the top of the dam is $10+15=25$ ft. The weight of the masonry, the uplift pressure, and the water pressure, and their moments about the back of the dam at the trial section are as follows:

Force	Amount	Arm	Moment
Weight of Masonry, W	$10\times25\times150=37{,}500$	$\frac{1}{2}\times10=5$	187,500
Uplift, U	$31.25\times.5\times10\times15=2{,}340$	$\frac{1}{3}\times10=3.33$	7,800
Water Pressure, T	$31.25\times15^2=7{,}030$	$\frac{1}{3}\times15=5$	35,200

The vertical component of the resultant force is

$$R_v=W-U=37{,}500-2{,}340=36{,}160 \text{ lb.}$$

and the resultant moment is

$$M=187{,}500-7{,}800+35{,}200=214{,}900 \text{ ft.-lb.}$$

Hence, the distance from the back of the dam to the resultant is

$$x=214{,}900\div35{,}160=6.11 \text{ ft.}$$

Since this distance is considerably less than $\frac{2}{3}\times10=6.67$ ft., the top of slope can be several feet below the trial position. If the depth below the top of the dam is taken as 28 ft., the calculations are:

Force	Amount	Arm	Moment
W	$10\times28\times150=42{,}000$	5	210,000
U	$31.25\times.5\times10\times18=2{,}810$	3.33	9,400
T	$31.25\times18^2=10{,}130$	6	60,800

$$R_v=42{,}000-2{,}810=39{,}190 \text{ lb.}$$

$$M=210{,}000-9{,}400+60{,}800=261{,}400 \text{ ft.-lb.}$$

$$x=261{,}400\div39{,}190=6.67 \text{ ft.}$$

Therefore, the distance 28 ft. is adopted.

Depths to Other Sections.—The sections at which the width of the dam is computed will be selected so that the distance below the preceding section is about 15 per cent. of the height of the dam above the section. The depths, in ft., to be used in this problem are 32, 37, 42, 48, 55, 63, 72, 83, 95, and 110.

Width at Depth 32.—The vertical distance from the top of slope, or the depth of 28 ft., to the section at a depth of 32 ft. is 4 ft. and the trial width at that section is taken as $10+\frac{1}{2}\times4=12$ ft. The shape of the upper part of the dam is as shown in Fig. 15, and the computations for level 32 are as follows:

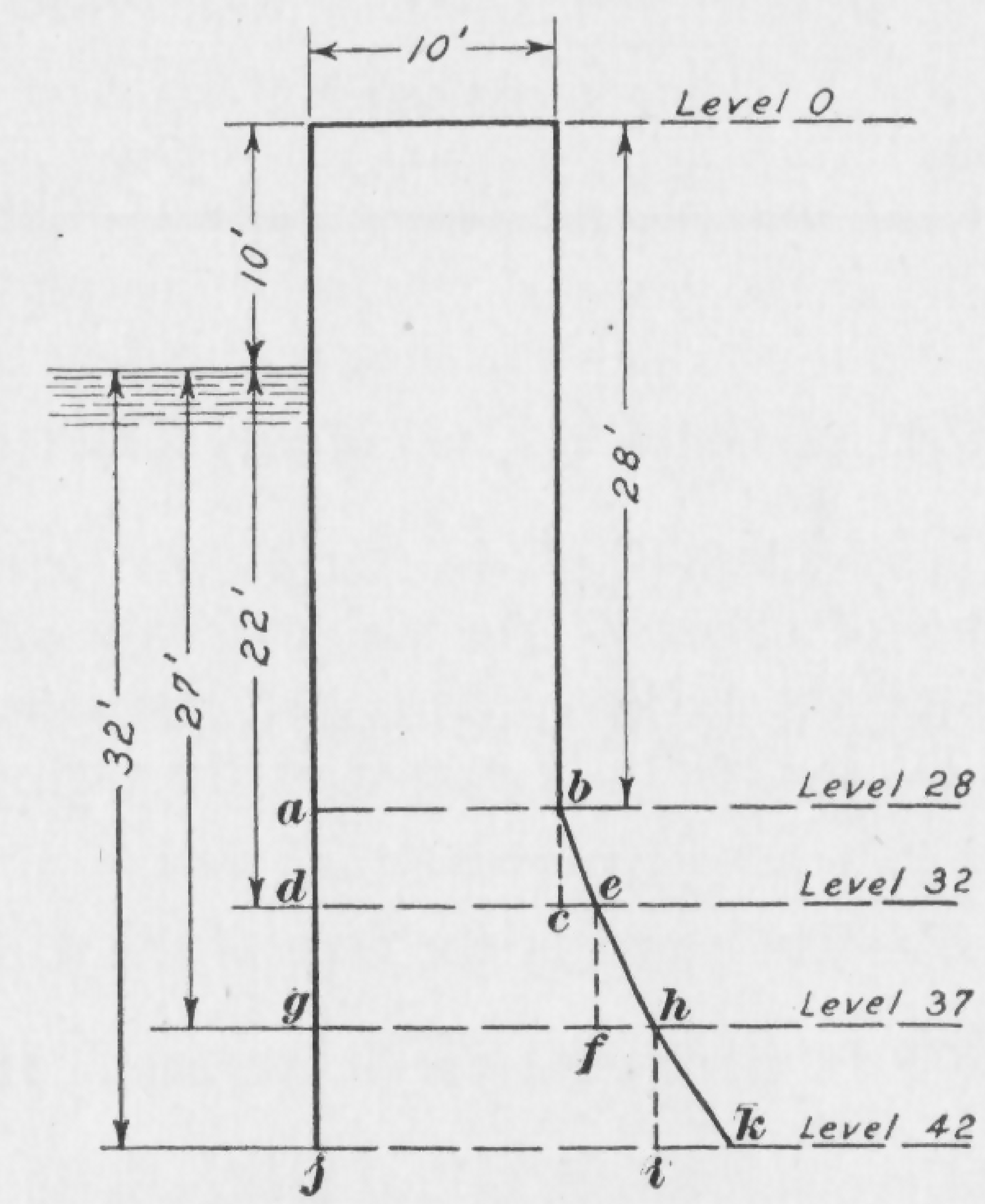

Fig. 15

Force	Amount	Arm	Moment
W_1 (above ab)	42,000	5	210,000
W_2 ($abcd$)	$10\times4\times150=6{,}000$	5	30,000
W_3 (bce)	$\frac{1}{2}\times2\times4\times150=600$	10.67	6,400
W (all masonry)	48,600		246,400
U	$31.25\times.5\times12\times22=4{,}100$	4	16,400
R_v	44,500		230,000
T	$31.25\times22^2=15{,}130$	7.33	110,900
			340,900

$$x=\frac{340{,}900}{44{,}500}=7.66 \text{ ft.}$$

Also, $\frac{2}{3}\times12=8$ ft., and the assumed width is too great. If the position of the resultant is assumed to remain unchanged, the new trial value of the width de should be $7.66\div\frac{2}{3}=11.49$, say 11.5 ft. For this dimension, the calculations follow:

Force	Amount		Arm	Moment
W_1		42,000	5	210,000
W_2		6,000	5	30,000
W_3	$\frac{1}{2}\times1.5\times4\times150$	= 450	10.5	4,700
W		48,450		244,700
U	$31.25\times.5\times11.5\times22=$	3,950	3.83	15,100
R_v		44,500		229,600
T		15,130	7.33	110,900
				340,500

Here, $x=\frac{340{,}500}{44{,}500}=7.65$ ft. and $\frac{2}{3}\times11.5=7.67$ ft. Therefore, the width of 11.5 ft. is adopted.

Obviously, the line of action of the weight of the masonry lies very close to the center of the section. Also, the resistance to sliding, or $44{,}500\times.75=33{,}380$ lb., is greatly in excess of the thrust T which tends to cause sliding.

Width at Level 37.—The slope of the face of the dam between levels 28 and 32 is $\frac{1.5}{4}=.375$ ft. horizontal per ft. vertical. If this slope continues to level 37, the required width at that section is $10+.375\times(37-28)=13.38$ ft., and this value will be used for the first trial. The shape of the dam above level 37 is as shown in Fig. 15. For the assumed width, the computations are:

Force	Amount		Arm	Moment
Masonry above *de*		48,450		244,700
W_4 (*defg*)	$5\times11.5\times150$	= 8,630	5.75	49,600
W_5 (*efh*)	$\frac{1}{2}\times1.88\times5\times150$	= 710	12.13	8,600
W		57,790		302,900
U	$31.25\times.5\times13.38\times27=$	5,640	4.46	25,200
R_v		52,150		277,700
T	31.25×27^2	=22,780	9	205,000
				482,700

In this case, $x=\frac{482{,}700}{52{,}150}=9.26$ ft. and $\frac{2}{3}\times13.38=8.92$ ft. Since the resultant passes outside of the middle third, the width of the dam must be increased. The next trial value will be taken as $9.26\div\frac{2}{3}=13.89$, or say 13.9, ft. The corrected computations are as follows:

Force	Amount		Arm	Moment
Masonry above *de*		48,450		244,700
W_4		8,630	5.75	49,600
W_5	$\frac{1}{2}\times2.4\times5\times150$	= 900	12.3	11,100
W		57,980		305,400
U	$31.25\times.5\times13.9\times27=$	5,860	4.63	27,100
R_v		52,120		278,300
T	31.25×27^2	=22,780	9	205,000
				483,300

Then, $x=\frac{483{,}300}{52{,}120}=9.27$ ft. and $\frac{2}{3}\times13.9=9.27$ ft. Since the position of the line of action of the weight of the masonry and the resistance to sliding are evidently satisfactory, the width of 13.9 ft. is adopted.

Width at Level 42.—The required width at a depth of 42 ft. is determined by proceeding as just described for the width at level 37. It is found that a width of 16.9 ft. is satisfactory, as shown by the following calculations:

Force	Amount		Arm	Moment
Masonry above *gh*		57,980		305,400
W_6 (*ghij*)	$5\times13.9\times150$	=10,420	6.95	72,400
W_7 (*hik*)	$\frac{1}{2}\times3\times5\times150$	= 1,130	14.9	16,800
W		69,530		394,600
U	$31.25\times.5\times16.9\times32=$	8,450	5.63	47,600
R_v		61,080		347,000
T	31.25×32^2	=32,000	10.67	341,400
				688,400

Hence, $x=\frac{688{,}400}{61{,}080}=11.27$ ft., which is just equal to $\frac{2}{3}\times16.9$. Also, the distance from the back of the dam to the line of action of the weight of the masonry is

$$z=\frac{394{,}600}{69{,}530}=5.68 \text{ ft.}$$

As this is slightly greater than $\frac{1}{3}\times16.9=5.63$ ft., no batter is required at the back of the dam at level 42.

Section at Level 48.—As the line of action of the weight of the dam passed so close to the edge of the middle third at level 42, it is very likely that the back of the dam will have to be battered at level 48.

Hence, for the calculations at this and the following sections, the reference line will be chosen 10 ft. beyond the back of the dam. For the first trial, the distance y from the reference line to the back of the dam will

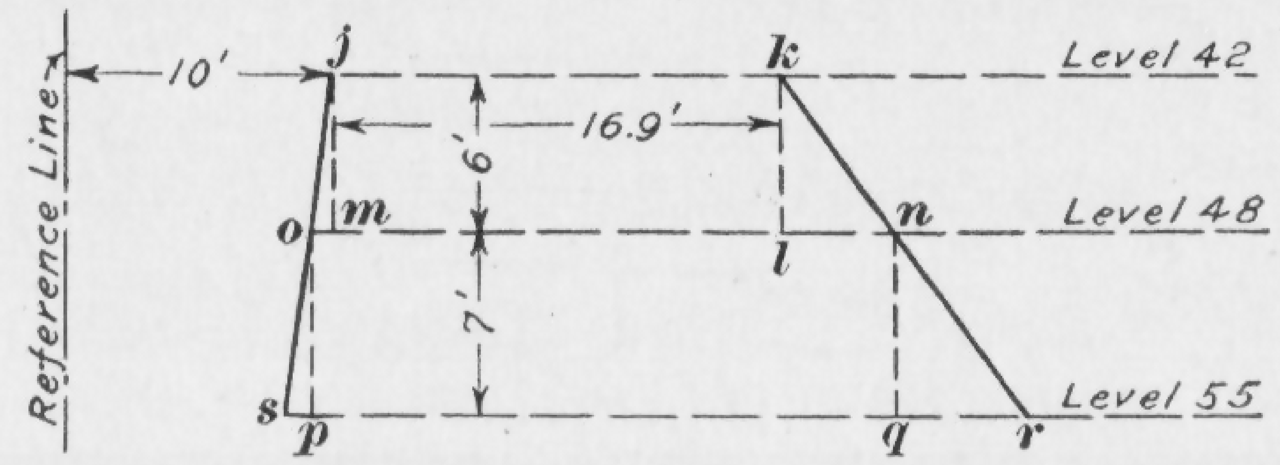

FIG. 16

be assumed as 9.5 and the width l of the section as 21.5 ft. The shape of the part of the dam between levels 42 and 55 is shown in Fig. 16. The calculations for level 48 follow, the arm for the moment of the masonry above jk being equal to 5.68+10=15.68 ft.

Force	Amount		Arm	Moment
Masonry above jk		69,530	15.68	1,090,000
W_8 $(jklm)$	$6\times16.9\times150$	=15,210	18.45	280,600
W_9 (kln)	$\frac{1}{2}\times4.1\times6\times150$	= 1,850	28.27	52,300
W_{10} (jmo)	$\frac{1}{2}\times.5\times6\times150$	= 220	9.83	2,200
W		86,810		1,425,100
Water on jo, V	$.5\times\left(\frac{32+38}{2}\right)\times62.5$	= 1,090	9.75	10,600
		87,900		1,435,700
U	$31.25\times.5\times21.5\times38$	=12,770	16.67	212,900
R_v		75,130		1,222,800
T	31.25×38^2	=45,130	12.67	571,800
				1,794,600

Here, $z=\frac{1,425,100}{86,810}=16.42$ ft. and $x=\frac{1,794,600}{75,130}=23.89$ ft. Also, $y+\frac{1}{3}l=9.5+\frac{1}{3}\times21.5=16.67$ ft. and $y+\frac{2}{3}l=23.83$ ft. Since z is less than $y+\frac{1}{3}l$ and x is greater than $y+\frac{2}{3}l$, both resultants lie outside of the middle third. Then, by the formulas of Art. 48,

$$l=3(x-z)=3\times(23.89-16.42)=22.41 \text{ ft.}$$

$$y=z-\tfrac{1}{3}l=16.42-\tfrac{1}{3}\times22.41=8.95 \text{ ft.}$$

Thus, the correct value of the width l lies between 21.5 and 22.41 ft. and of the distance y between 8.95 and 9.5 ft. It is found that a width of 22.0 ft. and a value of y of 9.08 ft. are satisfactory, as shown by the following calculations:

Force	Amount		Arm	Moment
Masonry above jk		69,530	15.68	1,090,000
W_8		15,210	18.45	280,600
W_9	$\frac{1}{2}\times4.18\times6\times150$	= 1,880	28.29	53,200
W_{10}	$\frac{1}{2}\times.92\times6\times150$	= 410	9.69	4,000
W		87,030		1,427,800
V	$.92\times\left(\frac{32+38}{2}\right)\times62.5$	= 2,010	9.54	19,200
		89,040		1,447,000
U	$31.25\times.5\times22\times38$	=13,060	16.41	214,300
R_v		75,980		1,232,700
T		45,130	12.67	571,700
				1,804,400

Hence, $z=\frac{1,427,800}{87,030}=16.41$ ft., which is equal to $y+\frac{1}{3}l=9.08+\frac{1}{3}\times22=16.41$ ft. Also, $x=\frac{1,804,400}{75,980}=23.75$ ft., which is equal to $y+\frac{2}{3}l=9.08+\frac{2}{3}\times22=23.75$ ft. The resistance to sliding, or $.75\times75,980=56,980$ lb., is much greater than the thrust exerted by the water, or 45,130 lb.

The maximum stress at the down-stream face is investigated as follows: In formula 1, Art. 30, $R_v=75,980$ lb., $b=22$ ft., and, since the resultant force passes through the edge of the middle third, $e=\frac{1}{6}b=\frac{22}{6}$. Then, the vertical unit pressure is

$$p=\frac{R_v}{b}\left(1+\frac{6e}{b}\right)=\frac{75,980}{22}\times\left(1+\frac{22}{22}\right)=6,910 \text{ lb. per sq. ft.}$$

Also, $\tan Y=\frac{4.18}{6}=.697$ and $Y=34°\ 53'$. Hence, by the formula of Art. 36, the maximum stress is

$$p'=p_m \sec^2 Y=6,910 \sec^2 34°\ 53'=10,270 \text{ lb. per sq. ft.}$$

which is considerably less than the allowable value of 40,000 lb.

Width at Level 55.—The computations for the required width and the projection at the back at each section between levels 55 and 110 are made in the same manner as shown for level 48. The values that are found to be satisfactory at level 55 are $y=8.1$ ft. and $l=28.1$ ft. For these dimensions, the calculations for investigating the position of the resultant are as follows:

Force	Amount		Arm	Moment
Masonry above *no*		87,030		1,427,800
W_{11} (*nopq*)	$7\times22\times150$	= 23,100	20.08	463,800
W_{12} (*nqr*)	$\frac{1}{2}\times5.12\times7\times150$	= 2,690	32.79	88,200
W_{13} (*ops*)	$\frac{1}{2}\times.98\times7\times150$	= 510	8.75	4,500
W		113,330		1,984,300
V_1 (on *jo*)		2,010	9.54	19,200
V_2 (on *os*)	$.98\times\left(\frac{38+45}{2}\right)\times62.5=$	2,540	8.59	21,800
		117,880		2,025,300
U	$31.25\times.5\times28.1\times45$	= 19,760	17.47	345,200
R_v		98,120		1,680,100
T	31.25×45^2	= 63,280	15	949,200
				2,629,300

$$z=\frac{1,984,300}{113,330}=17.51 \text{ ft. and } y+\tfrac{1}{3}l=8.1+9.37=17.47 \text{ ft.}$$

$$x=\frac{2,629,300}{98,120}=26.80 \text{ ft. and } y+\tfrac{2}{3}l=8.1+18.73=26.83 \text{ ft.}$$

Widths at Other Sections.—At a depth of 63 ft., it is found that y should be 7.4 ft. and l should be 34.7 ft., as shown by the following calculations:

Force	Amount		Arm	Moment
Masonry above 55 ft.		113,330		1,984,300
Masonry from 55 to 63	$8\times28.1\times150$	= 33,720	22.15	746,900
	$\frac{1}{2}\times5.9\times8\times150$	= 3,540	38.17	135,100
	$\frac{1}{2}\times.7\times8\times150$	= 420	7.87	3,300
W		151,010		2,869,600
V_1 (above 55)	$2,010+2,540$	= 4,550		41,000
V_2 (55 to 63)	$.7\left(\frac{45+53}{2}\right)\times62.5$	= 2,140	7.75	16,600
		157,700		2,927,200
U	$31.25\times.5\times34.7\times53$	= 28,730	18.97	545,000
R_v		128,970		2,382,200
T	31.25×53^2	= 87,780	17.67	1,551,100
				3,933,300

$$z=\frac{2,869,600}{151,010}=19.00 \text{ ft. and } y+\tfrac{1}{3}l=7.4+11.57=18.97 \text{ ft.}$$

$$x=\frac{3,933,300}{128,970}=30.50 \text{ ft. and } y+\tfrac{2}{3}l=7.4+23.13=30.53 \text{ ft.}$$

At a depth of 72 ft., the adopted values are $y=6.85$ ft. and $l=42.0$ ft. The calculations for this section follow:

Force	Amount		Arm	Moment
Masonry above 63 ft.		151,010		2,869,600
Masonry from 63 to 72	$9\times34.7\times150$	= 46,850	24.75	1,159,500
	$\frac{1}{2}\times6.75\times9\times150$	= 4,560	44.35	202,200
	$\frac{1}{2}\times.55\times9\times150$	= 370	7.22	2,700
W		202,790		4,234,000
V_1 (above 63)	$4,550+2,140$	= 6,690		57,600
V_2 (63 to 72)	$.55\times\left(\frac{53+62}{2}\right)\times62.5=$	1,980	7.12	14,100
		211,460		4,305,700
U	$31.25\times.5\times42\times62$	= 40,690	20.85	848,400
R_v		170,770		3,457,300
T	31.25×62^2	=120,130	20.67	2,483,100
				5,940,400

$$z=\frac{4,234,000}{202,790}=20.88 \text{ ft. and } y+\tfrac{1}{3}l=6.85+14=20.85 \text{ ft.}$$

$$x=\frac{5,940,400}{170,770}=34.79 \text{ ft. and } y+\tfrac{2}{3}l=6.85+28=34.85 \text{ ft.}$$

For the section at level 83 ft., satisfactory values are $y=6.45$ ft. and $l=50.6$ ft. The calculations are:

Force	Amount		Arm	Moment
Masonry above 72 ft.		202,790		4,234,000
Masonry from 72 to 83	$11\times42\times150$	= 69,300	27.85	1,930,000
	$\frac{1}{2}\times8.2\times11\times150$	= 6,770	51.58	349,200
	$\frac{1}{2}\times.4\times11\times150$	= 330	6.72	2,200
W		279,190		6,515,400
V_1 (above 72)	$6,690+1,980$	= 8,670		71,700
V_2 (72 to 83)	$.4\times\left(\frac{62+73}{2}\right)\times62.5=$	1,690	6.65	11,200
		289,550		6,598,300
U	$31.25\times.5\times50.6\times73$	= 57,720	23.32	1,346,000
R_v		231,830		5,252,300
T	31.25×73^2	=166,530	24.33	4,051,700
				9,304,000

$$z=\frac{6{,}515{,}400}{279{,}190}=23.34 \text{ ft. and } y+\tfrac{1}{3}l=6.45+16.87=23.32 \text{ ft.}$$

$$x=\frac{9{,}304{,}000}{231{,}830}=40.13 \text{ ft. and } y+\tfrac{2}{3}l=6.45+33.73=40.18 \text{ ft.}$$

At a depth of 95 ft., it is found that y should be 6.15 ft. and l should be 59.8 ft., as shown by the following calculations:

Force	Amount		Arm	Moment
Masonry above 83 ft.		279,190		6,515,400
Masonry from 83 to 95	$12\times50.6\times150$	= 91,080	31.75	2,891,800
	$\tfrac{1}{2}\times8.9\times12\times150$	= 8,010	60.02	480,800
	$\tfrac{1}{2}\times.3\times12\times150$	= 270	6.35	1,700
W		378,550		9,889,700
V_1 (above 83)	8,670+1,690	= 10,360		82,900
V_2 (83 to 95)	$.3\times\left(\frac{73+85}{2}\right)\times62.5$	= 1,480	6.30	9,300
		390,390		9,981,900
U	$31.25\times.5\times59.8\times85$	= 79,420	26.08	2,071,300
R_v		310,970		7,910,600
T	31.25×85^2	=225,780	28.33	6,396,300
				14,306,900

$$z=\frac{9{,}889{,}700}{378{,}550}=26.13 \text{ ft. and } y+\tfrac{1}{3}l=6.15+19.93=26.08 \text{ ft.}$$

$$x=\frac{14{,}306{,}900}{310{,}970}=46.00 \text{ ft. and } y+\tfrac{2}{3}l=6.15+39.87=46.02 \text{ ft.}$$

At the base of the dam or level 110, the adopted values are $y=5.9$ ft. and $l=71.2$ ft. The calculations for this section are as follows:

Force	Amount		Arm	Moment
Masonry above 95 ft.		378,550		9,889,700
Masonry from 95 to 110	$15\times59.8\times150$	=134,550	36.05	4,850,500
	$\tfrac{1}{2}\times11.15\times15\times150$	= 12,540	69.67	873,700
	$\tfrac{1}{2}\times.25\times15\times150$	= 280	6.07	1,700
W		525,920		15,615,600
V_1 (above 95)	10,360+1,480	= 11,840		92,200
V_2 (95 to 110)	$.25\times\left(\frac{85+100}{2}\right)\times62.5$	= 1,450	6.03	8,700
		539,210		15,716,500
U	$31.25\times.5\times71.2\times100$	=111,250	29.63	3,296,300
R_v		427,960		12,420,200
T	31.25×100^2	=312,500	33.33	10,416,700
				22,836,900

$$z=\frac{15{,}615{,}600}{525{,}920}=29.69 \text{ ft. and } y+\tfrac{1}{3}l=5.9+23.73=29.63 \text{ ft.}$$

$$x=\frac{22{,}836{,}900}{427{,}960}=53.36 \text{ ft. and } y+\tfrac{2}{3}l=5.9+47.47=53.37 \text{ ft.}$$

Also, the resistance to sliding, which is $.75\times427{,}960=321{,}000$ lb., exceeds the thrust T, or 312,500 lb.

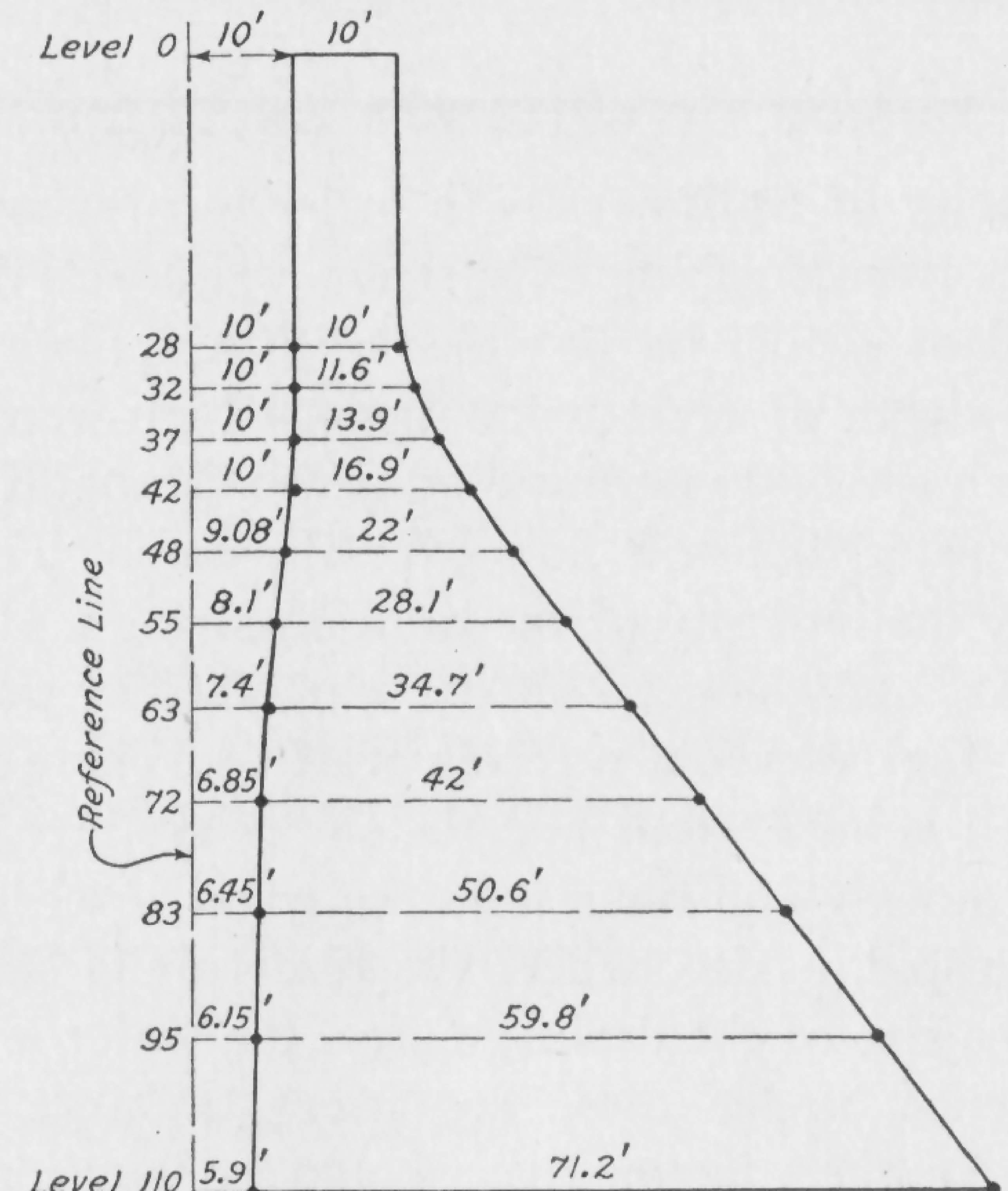

Fig. 17

Compressive Stresses.—For a dam of this height, the compressive stresses are well within the allowable values as shown by the following calculations at the base. In formula 1, Art. 30, $R_v=427{,}960$ lb., $b=71.2$ ft., and $e=x-(y+\tfrac{1}{2}l)=53.36-(5.9+35.6)=11.86$ ft. Then, the maximum vertical unit pressure at the face of the dam is

$$p=\frac{R_v}{b}\left(1+\frac{6e}{b}\right)=\frac{427{,}960}{71.2}\times\left(1+\frac{6\times11.86}{71.2}\right)=12{,}020 \text{ lb. per sq. ft.}$$

Also, the angle Y that the face of the dam makes with the vertical is found from the relation $\tan Y=\frac{11.15}{15}=.743$. Hence, $Y=36°\ 37'$ and, by the formula of Art. 36, the maximum inclined unit stress is

$$p'=p_m \sec^2 Y=12{,}020 \sec^2 36°\ 37'=18{,}660 \text{ lb. per sq. ft.}$$

This is considerably less than the allowable value of 40,000 lb.

Cross-Section of Dam.—In selecting the final section of the dam, the first step is to plot at each level considered in the design the points on the up-stream and down-stream faces that are determined by the calculated values of y and l. In Fig. 17 the points marked by the small circles are located in accordance with the values of y and l computed in the preceding design. Smooth curves are then drawn for the face and back of the adopted section, care being taken to keep the final surface lines outside of the theoretical points.

DESIGN OF SPILLWAY SECTIONS

51. Capacity of Spillways.—In order to prevent the water behind a dam from reaching excessively high levels during flood stages, the dam should be provided with a spillway of ample dimensions—depth of crest below normal high-water level and length of such crest—to permit readily the escape of the largest quantity of water that will flow over the dam. This quantity is affected by the intensity of the rainfall during a violent storm and by the size and character of the watershed or drainage area from which the reservoir receives water. Where the soil of the watershed is impervious and the slopes are steep, the water will run off quickly and the rise in the water level in the reservoir will be rapid. Also, where the reservoir is small in comparison to the size of the drainage area, the water level will rise rapidly. Hence, under these conditions, a large spillway is necessary. Another factor that should be considered in the design of the spillway is the possible damage that would result from failure of the dam.

Valuable data on past flood flows may sometimes be obtained by consulting local inhabitants. Observations and records of flood flows at other dams located on streams having similar characteristics may also serve as a good basis for estimating future flood flows for the dam under consideration. General formulas for run-off are seldom reliable as a basis in design, as they are usually derived from local conditions and do not apply accurately under different circumstances. When no definite information in regard to the flood flow is available, it is common practice in the case of watersheds of moderate size—20 to 200 square miles in area—to provide spillways that are large enough to discharge in 24 hours a quantity of water that would cover the entire area of the watershed to a depth of 6 inches. Spillways for small watersheds have been designed to discharge the quantity that would stand to a depth of 8 inches. For example, if the area of a watershed is 7.5 square miles and the spillway is to take care of a depth of 8 inches on this area, the required capacity of the spillway is $7.5\times5{,}280\times5{,}280\times\frac{8}{12}=139{,}392{,}000$ cubic feet per day or $139{,}392{,}000\div86{,}400=1{,}613$ cubic feet per second.

52. Dimensions of Spillway.—After the required capacity of the spillway has been determined, the next step is to establish the length of the spillway and the vertical distance from the high-water level to the crest of the spillway. There is no general rule for fixing either the length or the depth of the spillway. The length of the dam, the width of the top, the presence or absence of a roadway on it, and local conditions in regard to flood flow all exert an influence on the relative dimensions of the spillway. Where no special considerations enter into the problem, the approximate length of the spillway is often determined by the formula

$$L=20\sqrt{A} \qquad (1)$$

in which L = length of spillway, in feet;

A = area of watershed above dam, in square miles.

When the distance from the crest of the spillway to high-water level is fixed, the required length L is found by the formula

$$L=\frac{Q}{3.4H^{\frac{3}{2}}} \qquad (2)$$

in which Q = required capacity of spillway, in cubic feet per second;

H = maximum head of water on crest of spillway, in feet.

In case the length of spillway is fixed by the conditions of the design, the corresponding required head H can be computed by the formula

$$H=\sqrt[3]{\left(\frac{Q}{3.4\,L}\right)^{2}} \qquad (3)$$

EXAMPLE.—If there are no limitations, what would be reasonable dimensions for the spillway of a dam for the watershed area considered in the preceding article?

SOLUTION.—Here the watershed area is 7.5 sq. mi., and, by formula 1, the approximate length of the spillway is

$$L=20\sqrt{A}=20\times\sqrt{7.5}=54.8 \text{ ft.}$$

The length of the spillway would probably be made between 50 and 60 ft. Ans.

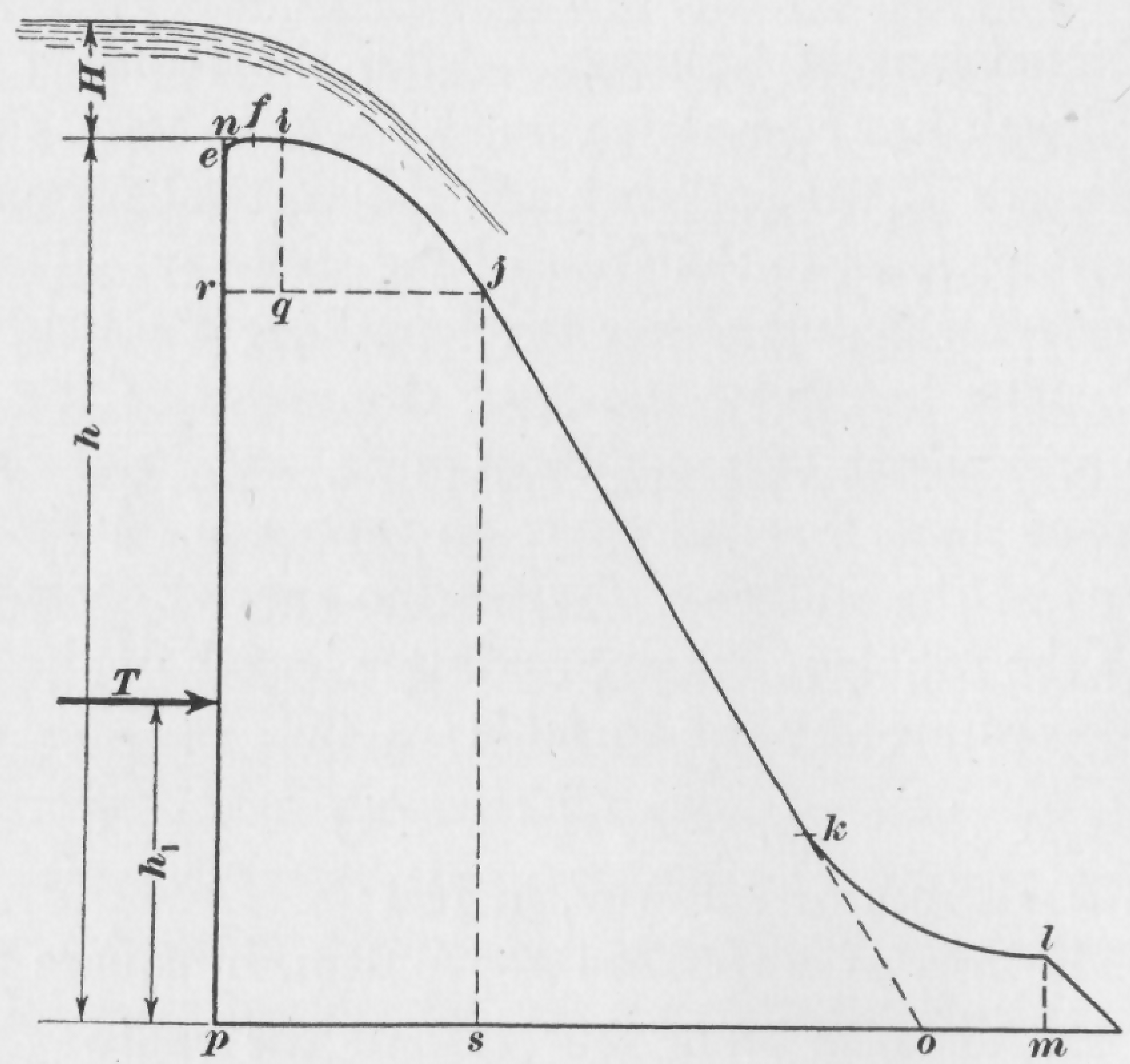

FIG. 18

Also, $Q=1{,}613$ cu. ft. per sec. and, if L is assumed to be 60 ft., the corresponding required head H on the spillway would, by formula 3, be

$$H=\sqrt[3]{\left(\frac{Q}{3.4L}\right)^2}=\sqrt[3]{\left(\frac{1{,}613}{3.4\times 60}\right)^2}=3.97, \text{ say 4, ft. Ans.}$$

53. Water Pressure on Spillway Section.—In Fig. 18 is shown a typical spillway section of a dam with water flowing over the crest. The horizontal thrust exerted by the water on the back of the dam above any level can be found by the formula

$$T_x=31.25\, h_x(h_x+2\,H) \qquad (1)$$

in which T_x = total horizontal pressure per linear foot of dam above any level, in pounds;
h_x = vertical distance from crest of spillway to level under consideration, in feet;
H = height of water above crest of spillway, in feet.

Also, the vertical distance h_p from the level in question to the point of application of the resultant water pressure can be determined by the formula

$$h_p=\tfrac{1}{3}\,h_x\left(\frac{h_x+3\,H}{h_x+2\,H}\right) \qquad (2)$$

For the section at the base of the dam, at which $h_x=h$ and $h_p=h_1$, the formulas are

$$T=31.25\,h(h+2\,H) \qquad (3)$$

$$h_1=\tfrac{1}{3}\,h\left(\frac{h+3\,H}{h+2\,H}\right) \qquad (4)$$

54. Shape of Masonry at Crest of Spillway.—The sheet of water that flows over the spillway crest is shaped as indicated in Fig. 19. In the illustration the water is shown discharging over a board a with a sharp edge, but the flow over the crest of a spillway like that in Fig. 18 is quite similar. The upper surface bcd of the curved sheet is known as the *upper nappe* and the lower surface efg as the *lower nappe*.

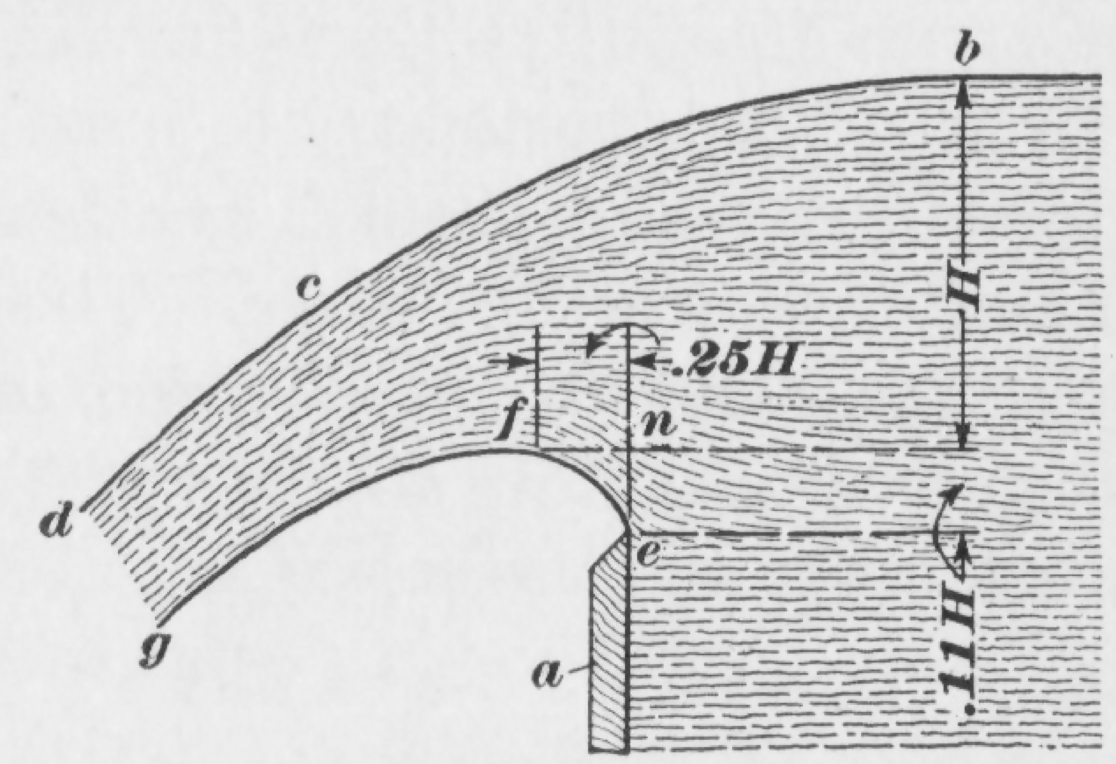

FIG. 19

In a well-designed spillway, the surface of the masonry is shaped to conform to the particular lower nappe that is produced by the greatest volume of water expected to flow over the spillway. When the spillway is shaped in that manner, the water is always in contact with the face of the dam, and the formation of a partial vacuum between the lower nappe and the face of the dam is thus prevented. Where the dam is comparatively short and the head on the crest of the over-

flow section is not very great, it may be feasible to prevent the formation of a vacuum by providing free access of air to the space between the face of the dam and the sheet of falling water. The air may be allowed to enter this space at the ends of the dam and at any intermediate piers which may serve to interrupt the continuous length of crest. If a space is left between the face of the spillway and the lower nappe and the outside air does not have free access to this space, vibration or trembling of the dam will result.

When the back of the dam is vertical, or nearly so, just below the top of the spillway, as is usual in the case of a gravity dam, the vertical distance from e to f in Fig. 19 may be taken as .11 H and the horizontal distance between these two points as .25 H, where H is the head on the crest of the spillway. The theoretical curve of the lower nappe beyond f is a parabola, whose equation is

$$x^2 = 1.78\,Hy \qquad (1)$$

in which x = horizontal distance from f to any point on lower nappe;

y = vertical distance from f to same point.

However, it is customary to make the crest of the spillway flat for a distance of about 2 feet beyond f, as shown in Fig. 18, and then to start the curve. This flat portion serves as a safe walkway along the crest. Also, in order to make sure that the water will not leave the face of the spillway, a more desirable equation of the parabola ij is

$$x^2 = 2.25\,Hy \qquad (2)$$

In some dams, the portion between e and f is made straight instead of curved.

55. Since the water surface is above the crest of the spillway, it is not possible to design the top of the spillway section so as to satisfy the theoretical requirements of stability. No matter how wide the top of the spillway is made, there will be a region near the top in which the resultant force will pass outside of the middle third of the masonry and the horizontal thrust will exceed the frictional resistance to sliding. However, where ice pressure is neglected, the tensile unit stresses produced at the back of the dam are usually less than the tensile strength of monolithic concrete. Hence, if construction joints are avoided in the region where tensile stresses occur, the shape of the crest may be established as explained in the preceding article. The shearing strength of the concrete is also sufficient to resist sliding.

Where ice pressure must be resisted, the tensile stresses at the back of the dam may be great for a considerable depth. In such cases, it is customary to provide steel reinforcement near the back of the dam in the region where tensile stresses are liable to occur. The amount of steel is determined by designing the structure as a reinforced-concrete cantilever to resist the bending moment due to the horizontal thrust of the ice and water pressure. In determining this bending moment, the weight of the concrete and the uplift pressure are disregarded. The steel rods also help to resist sliding.

The positions of the lowest levels at which tensile stresses may be permitted and at which the horizontal thrust may exceed the frictional resistance depend on the conditions in the particular case and on the judgment of the designer.

56. Protection of Foundation.—When the water flowing over the spillway is allowed to follow its natural path, it falls nearly vertically as it strikes the tail-water. Where the drop is comparatively small, say less than about 20 or 30 feet, and the foundation bed is of solid rock, the shock produced by the falling water is not dangerous unless logs or large blocks of ice pass over the spillway. Hence, in designing the spillways of some low dams, no special provisions were made for protecting the foundation bed from the force of the falling water. Where the fall is great, the water attains considerable velocity in its drop and, if it is permitted to strike the foundation bed vertically, even a bed of solid rock would soon be worn away and the dam would be undermined.

Sometimes a dam already exists below the proposed dam, and it may then be assumed that the reservoir behind the existing dam will form a permanent tail-water that will act as

a cushion for the water flowing over the spillway of the proposed dam and will thus protect the foundation from scour. Such a cushion absorbs much of the energy of the falling water and also reduces the height of the drop, so that the water flowing over the spillway may be allowed to follow its natural path. However, if there is no permanent tail-water, it is necessary to reduce the shock and scouring effect of the falling water by shaping the face of the spillway so that the direction of the water is gradually altered until it is horizontal at the discharge end, as shown in Fig. 18. The curved portion *kl,* by means of which the water is gradually deflected at the bottom of the spillway, is known as the bucket. Even when a bucket is provided, there is liable to be considerable scouring action on the foundation where the drop is great, unless there is also a water cushion or the velocity of the water is retarded by an obstruction, such as a small secondary dam placed just beyond the toe of the spillway. The height of the drop can often be reduced by placing the spillway near one end of the dam.

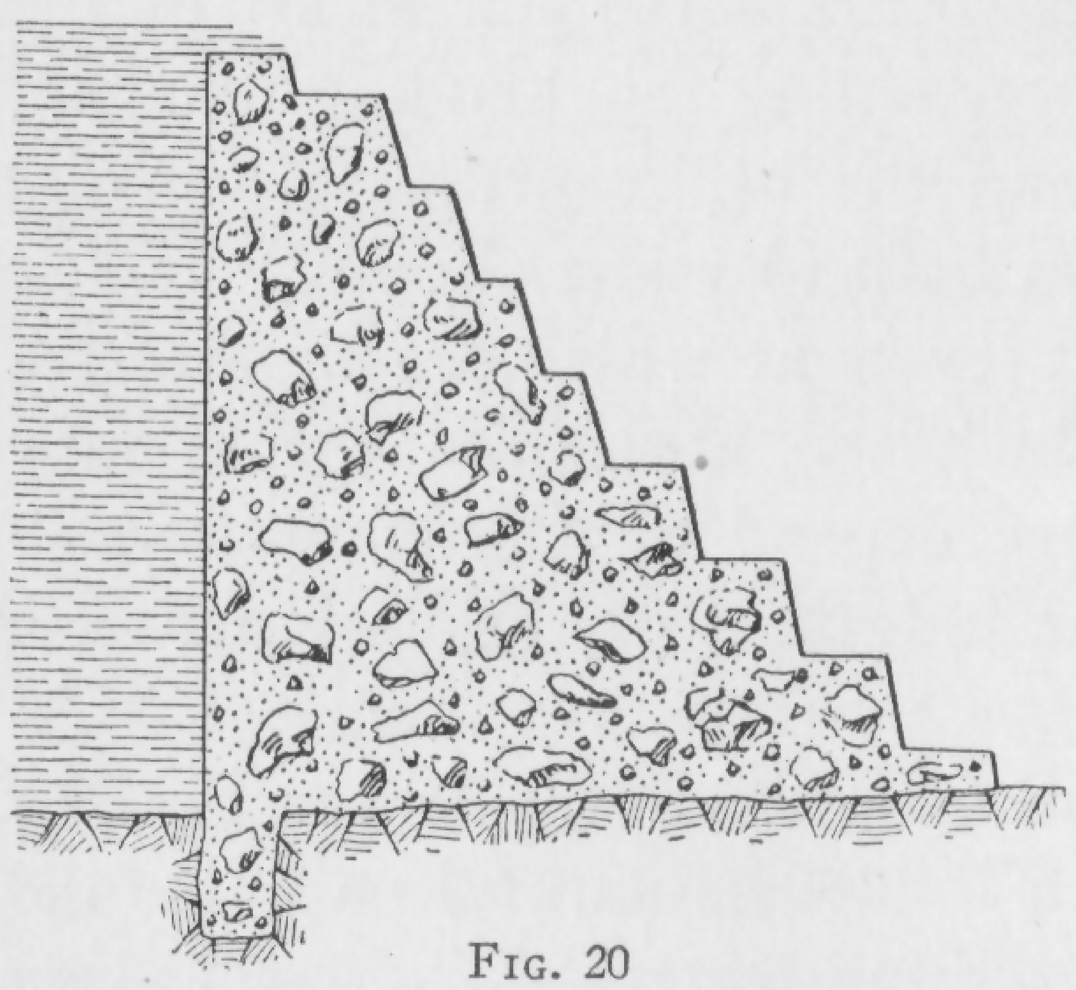

FIG. 20

The curve of the bucket is often made tangent to the foundation, but where a water cushion is provided the point *l,* Fig. 18, should be at the lowest probable level of the tail-water surface. In either case, the curve of the bucket is generally a circular arc whose radius is between $\frac{1}{4}h$ and $\frac{1}{3}h$, where h is the height of the spillway section. Below *l,* the face of the spillway may be vertical, as shown by the dotted line *lm,* but a slope of about 45° is preferable. The **S**-shaped curve *ijkl* is often referred to as an ogee curve, and a section of a dam having this form is called an ogee section.

57. Stepped Spillways.—In some cases the face of the dam is stepped, as shown in Fig. 20. The purpose of the stepping is to break up the force of the water, whereas a curved face facilitates the removal of the water. The use of the stepped face is quite generally restricted to low dams, or to dams where overflows are relatively small.

58. Method of Designing Spillway Section.—The general method of procedure in determining the required width of the dam at any depth of a spillway section is the same as that described for a non-overflow section. However, as previously explained, the shape of the crest requires special consideration, and a bucket is usually introduced near the bottom of the dam. In calculating the weight and moment of the masonry, the curves may be considered replaced by straight lines. For instance, in Fig. 18, a straight line from *i* to *j* may be substituted for the curve joining those two points. Likewise, the omission of the small amount of masonry caused by cutting off the corner between *e* and *f* is disregarded and the dam is treated as if there was a square corner, as shown by the dotted lines *en* and *fn.* The weight of the water above the spillway section and the effect of friction between the flowing water and the masonry surface may also be neglected. Thus, if it were required to find the width of a horizontal section below *j,* the forces to be considered would be the horizontal pressure T_x exerted by the water on the back of the dam above that section, whose amount and position are determined by formulas 1 and 2, Art. 53; the weight of the masonry above the section; the uplift pressure, the head at the back of the dam being equal to the distance from the water surface to the section; and, if the back of the dam is inclined above the section, the vertical component of the water pressure on the inclined part.

When ice pressure need not be considered, a satisfactory section is generally obtained by shaping the top of the spillway as described in Art. 54, and sloping the face uniformly between the end of the parabola at the top and the bucket at the bottom, as between the points *j* and *k* in Fig. 18. To determine the slope of *jk,* it is customary to establish by trial the required width *op* at the base of the cross-section and to make the line *jo* tangent to the parabola *ij* at *j.* In selecting the first trial sec-

tion, the distance op may be made equal to $.7(h+H)$. The point j can be located best by drawing the section to scale. In case the assumed width op is not satisfactory, a new width is selected by judgment, the points o and j are relocated, and the section thus established is investigated. After the position of the line oj has been fixed, the curve kl is drawn outside that line, as shown in the illustration.

Where provision for ice pressure is necessary, it may be advisable to have the crest of the spillway outside of the parabolic curve determined as in Art. 54. In that case, the required widths at various levels are determined by trial, as in the case of a non-overflow section, and the adopted section is established by judgment.

EXAMPLE.—It is required to design a spillway section similar to that shown in Fig. 18. The height h is 60 feet and the maximum head H on the crest of the spillway is 8 feet. Tail-water pressure is neglected.

SOLUTION.—As recommended in Art. 54, the horizontal distance from the back of the dam to f is taken as $.25\,H=.25\times 8=2$ ft. and the vertical distance from e to f as $.11\,H=.11\times 8=.88$ ft. Also, the flat width fi will be made 2 ft.

The equation of the parabola ij with respect to its vertex i as the origin of coordinates is

$$x^2=2.25\,Hy=2.25\times 8\,y=18\,y$$

The curve ij is then plotted from this equation by locating three points corresponding to values of x of 5, 10, and 15, respectively. Thus, for $x=5$,

$$25=18\,y,\ \text{and}\ y=1.39\ \text{ft.}$$

For $x=10$,

$$100=18\,y,\ \text{and}\ y=5.56\ \text{ft.}$$

For $x=15$,

$$225=18\,y\ \text{and}\ y=12.5\ \text{ft.}$$

Also, the distance op on the trial section is taken as $.7\times(60+8)=47.6$ or say 48 ft. By drawing a line from o tangent to the curve ij, it is found that point j is about 10 ft. below the crest of the spillway. At this height, the horizontal distance from i to j is, from the equation $x^2=18\,y$,

$$x=\sqrt{18\times 10}=13.42\ \text{ft.}$$

and the width of the dam is $2+2+13.42=17.42$ ft. If the face of the dam is assumed to be straight from i to j, and the back of the dam is taken as the reference line for moments, the calculations for the width op are as follows:

Force	Amount		Arm	Moment
W_1 $(niqr)$	$10\times 4\times 150$	$=6{,}000$	2	12,000
W_2 (ijq)	$\frac{1}{2}\times 13.42\times 10\times 150$	$=10{,}070$	8.47	85,300
W_3 $(rjsp)$	$17.42\times 50\times 150$	$=130{,}650$	8.71	1,138,000
W_4 (jos)	$\frac{1}{2}\times 30.58\times 50\times 150$	$=114{,}670$	27.61	3,166,000
W		$=261{,}390$		4,401,300
U	$31.25\times .5\times 48\times 68$	$=51{,}000$	16	816,000
R_v		$=210{,}390$		3,585,300
T	$31.25\times 60\times(60+2\times 8)$	$=142{,}500$	22.11	3,150,700
				6,736,000

Here $$z=\frac{4{,}401{,}300}{261{,}390}=16.84\ \text{ft., whereas}\ \tfrac{1}{3}\times 48=16\ \text{ft.}$$

Also, $$x=\frac{6{,}736{,}000}{210{,}390}=32.02\ \text{ft., whereas}\ \tfrac{2}{3}\times 48=32\ \text{ft.}$$

Although the resultant of all the forces cuts the base .02 ft. outside the middle third of the theoretical section, the section need not be changed because the effect of the bucket and the tail-water pressure will be to increase the stability. Also, it will be found that the width rj is sufficient. Hence, the assumed position of jo may be considered satisfactory.

If the radius of the curve kl is taken as $\frac{1}{3}\,h$, it will be $\frac{1}{3}\times 60=20$ ft. This curve will be made tangent to the line jo, the position of the point k on that line being determined by the required elevation of the point l at which the curve becomes horizontal.

EXAMPLES FOR PRACTICE

1. If the area of the watershed above a proposed dam is 40 square miles, what would be the required capacity of the spillway in cubic feet per second, according to common practice? Ans. 6,450 cu. ft. per sec.

2. (*a*) If there are no special features to be considered, what would be the approximate length of the spillway in the preceding example? (*b*) If the length of spillway is made 130 feet, what will be the probable maximum depth of water above the crest?

$$\text{Ans.}\begin{cases}(a)\ 126.5\ \text{ft.}\\(b)\ 5.97,\ \text{say}\ 6\ \text{ft.}\end{cases}$$

3. In a spillway having a section like that shown in Fig. 18, the height h is 75 feet, the height H is 10 feet, the distance nf is made equal to $.25\,H$, fi is made 2.5 feet, and the equation of the parabola ij is assumed to be $x^2=2.25\,Hy$. If ice pressure can be neglected, determine (*a*) the trial width op, (*b*) the vertical distance from the crest fi to the point j at which the line oj is tangent to the curve ij, and (*c*) the width rj of the dam at that level.

$$\text{Ans.}\begin{cases}(a)\ 60\ \text{ft.}\\(b)\ 17\ \text{ft.}\\(c)\ 24.56\ \text{ft.}\end{cases}$$

4. Using the various distances that are specified and calculated in the preceding example, determine the horizontal distance from the heel p to the point where the resultant of all the forces cuts the base po.

Ans. 40.22 ft.

ARCHED GRAVITY SECTIONS

59. When a high dam is to be built in a gorge with rocky sides, the arched gravity type is usually most suitable. In the case of a dam built on a curve, any two cross-sections converge toward the center of the curve, and the line joining the centers of gravity of the sections is shorter than the distance between the sections measured along the back of the dam. Therefore, the weight of masonry between two sections of a curved dam that are a certain distance apart on the back is less than for a straight dam. On the other hand, in a dam that arches up-stream, the curved section resists failure partly by arch action and it is generally conceded that, where a reasonable degree of curvature is obtained and suitable abutments are provided, a curved dam is more stable than a straight dam having the same cross-section. Nevertheless, the difference in stability between a straight dam and a curved dam is usually disregarded, and the section of a curved gravity dam is generally determined as if the dam were straight across the stream.

ARCH AND HOLLOW DAMS

ARCH DAMS

60. Methods of Design.—The function of an arch dam is to transmit the water pressure by arch action to the walls of the canyon that serve as its abutments. Therefore, the stability of such a dam, unlike that of a gravity dam, depends on the crushing strength of the material and not on the weight. It is very difficult to make an exact mathematical analysis of the stresses produced in an arch dam, because the theory underlying the design of arches is rather complex and a number of indeterminate factors enter in the case of arch dams. In a simple method that has received considerable application, the dam is treated as if composed of a series of independent

horizontal layers or rings, each of which is part of a thin cylindrical shell and is subjected only to the crushing effect of the horizontal component of the water pressure. No dam designed by this method has ever failed. But, experiments conducted on the Stevenson Creek test dam in California by a committee of the Engineering Foundation indicated that stresses produced in dams so designed are greater than those generally considered desirable.

In the more accurate methods of design for arch dams that are now in use, due consideration is given to the facts that the arch rings are comparatively thick and are generally fixed at their supports, that the pressure of the water tends not only to crush the arch rings but also to overturn the dam, and that important stresses are produced because of temperature changes and shrinkage in the concrete and other factors. However, all such accurate investigations must be based on certain tentative dimensions for the various sections of the dam. When the stresses are computed, the dimensions are modified as found necessary and another stress analysis is made. This procedure is continued until the unit stresses in the various parts of the dam are within desirable limits. In the so-called *trial-load method,* developed by the United States Bureau of Reclamation, tentative dimensions for the dam are first selected and the dam is assumed to be divided into a system of horizontal arches fixed at their supports and a system of vertical cantilevers fixed at their lower ends. After a series of trials and recomputations, the water and temperature loads are distributed among the arches and cantilevers so that at any intersection of a cantilever with an arch the deflection along the radius produced by the loading on the cantilever will practically be equal to the deflection caused by the loading on the arch. As in other comparatively accurate methods, the tentative dimensions are modified in accordance with the computed stresses, and further investigations are made until a satisfactory design is obtained.

61. Use of Models.—It has been demonstrated that fairly accurate predictions of the deflections and stresses in an arch dam can be obtained by means of tests on a miniature model of

it made of celluloid, hard rubber, or concrete. Thus, the deflections and stresses observed in testing the Stevenson Creek dam agreed closely with those predicted by Professor Beggs as a result of his tests on a celluloid model—one-fortieth the size of the actual test dam—that was loaded by him with mercury. Hence, after the dimensions of a dam have been established by means of theoretical considerations, a good check of the design may be made by testing a model of the dam.

62. **Cylinder Formula.**—In selecting the tentative dimensions for the analysis of an arch dam, it is convenient to assume that the dam is subjected only to the crushing effect of the horizontal component of the water pressure and is divided into horizontal rings that are parts of thin cylinders. As previously stated, the design of some arch dams has been based entirely on this approximate method. In Fig. 21 is shown a horizontal section of an arch dam. The faces of the dam, *ab* and *cd*, are arcs of circles, the two arcs having the common center *o*. When the dam is treated as a segment of a thin cylindrical shell under the action of an external normal pressure, the thickness at any elevation may be found by the following cylinder formula:

$$t=\frac{Rp}{f}$$

in which t = required thickness of arch, in feet;
R = radius of outside, or up-stream, face of dam, in feet;
p = water pressure at level under consideration, in pounds per square foot;
f = permissible compressive unit stress in masonry, in pounds per square foot.

In concrete arch dams designed on the basis of arch action alone, the value of f has usually been taken between 40,000 and 60,000 pounds per square foot. This is considerably below the allowable unit stress of 90,000 pounds per square foot, which is frequently used in designing arch dams by more accurate methods.

EXAMPLE.—At a depth of 40 feet below the water level, the radius of the up-stream face of an arch dam is to be 300 feet. If only arch action is considered and the cylinder formula is applied, in which the allowable stress is taken as 50,000 pounds per square foot, what is the required thickness of the arch at that depth?

SOLUTION.—Here $R=300$ ft., $p=62.5\times40=2{,}500$ lb. per sq. ft. and $f=50{,}000$ lb. per sq. ft. Therefore, the required thickness is

$$t=\frac{Rp}{f}=\frac{300\times2{,}500}{50{,}000}=15 \text{ ft. Ans.}$$

63. **Constant-Radius Dams.**—There are two general classes of arch dams, namely, constant-radius dams and constant-angle dams. As the name implies, all horizontal sections of a con-

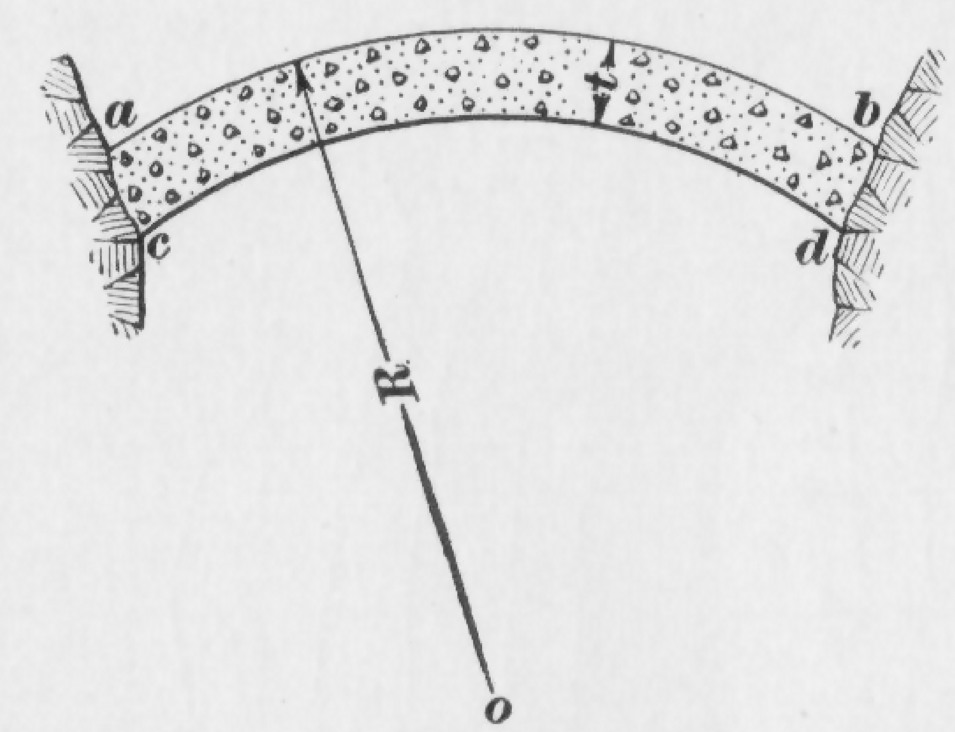

FIG. 21

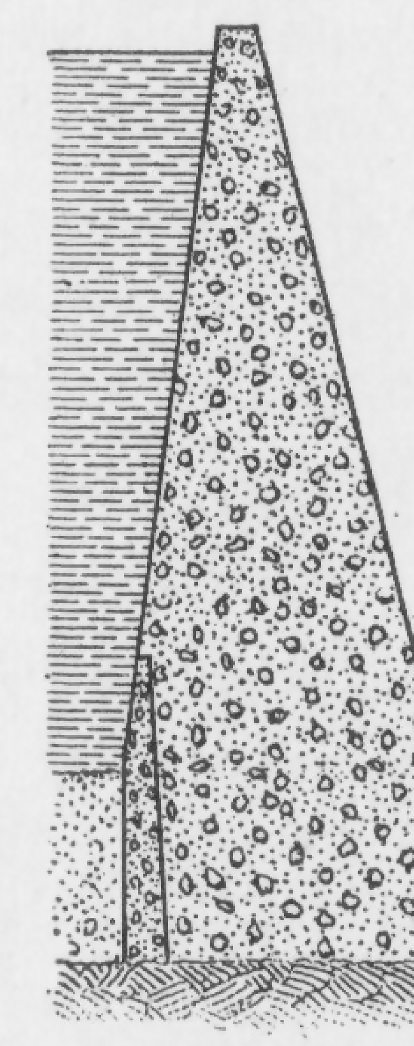

FIG. 22

stant-radius dam have the same radius of curvature. Hence, according to the cylinder formula, the required thickness of the masonry increases in direct proportion to the height. In low dams of the constant-radius type, the up-stream face is usually vertical and the down-stream face slopes uniformly from the top, or from a level near the top, to the bottom. In high dams, both faces are generally sloped, as shown in Fig. 22. Since the gorge in which the dam is constructed becomes narrower toward the bottom, the length of the dam is smaller at the lower than at the upper sections. As a result, the central angle between the radii drawn to the extremities of the section at any level likewise becomes less towards the bottom of the dam and

the lower portions are more like straight beams than arches. Therefore, arch action cannot be considered fully effective, and the lower part of the dam must be made quite thick in order to resist the water pressure safely.

64. Constant-Angle Dams.—In the case of a constant-angle dam, the central angle between the radii at the extremities of every horizontal section is the same, and the arch action is therefore just as effective in the lower portion of the dam as in the upper portion. Hence, the lower portion of a constant-angle dam can be made much thinner than that of a constant-radius

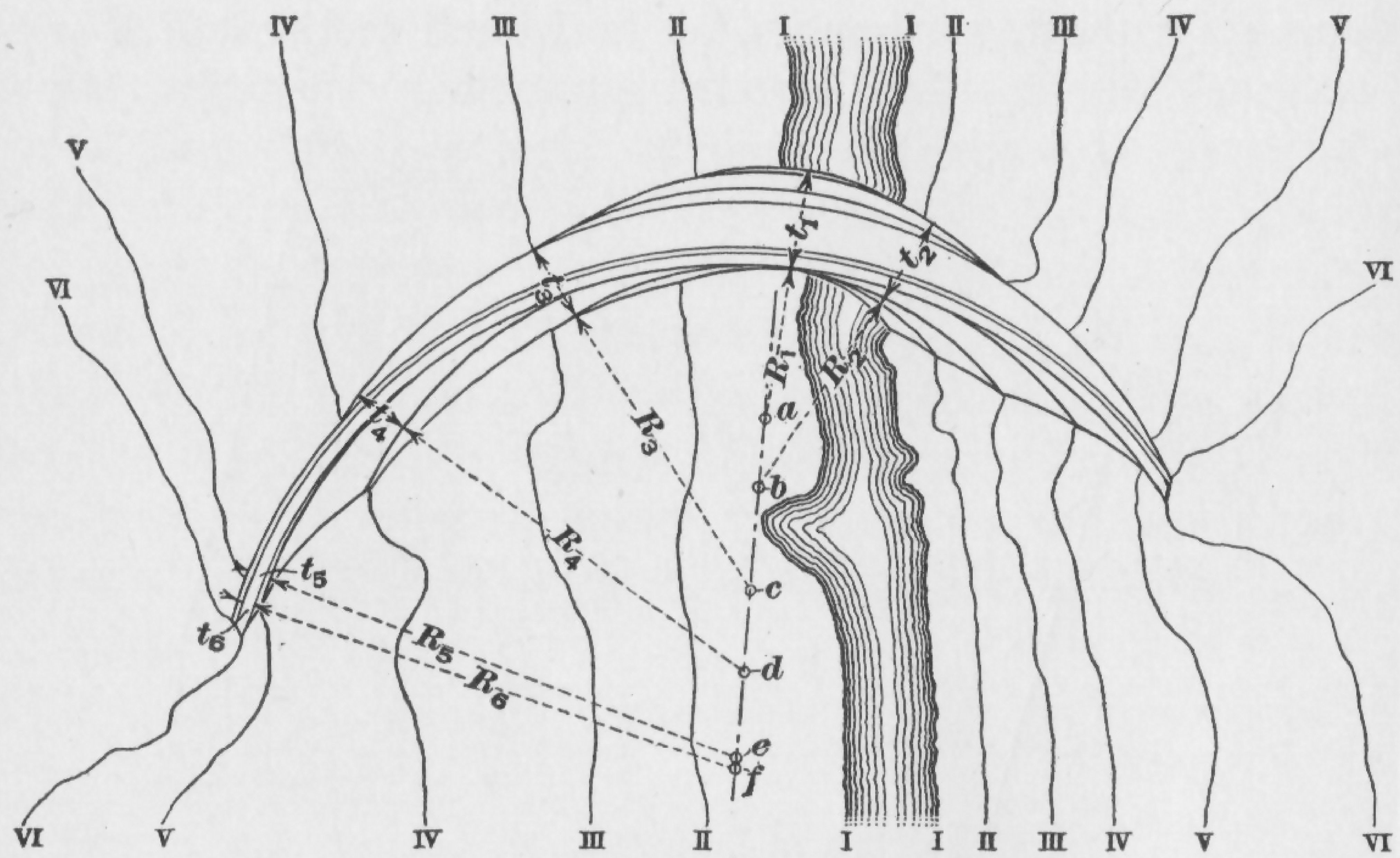

Fig. 23

dam of the same height. Theoretically, the most economical cross-section for an arch dam is obtained when the central angle for every horizontal section is 133°; but, actually, there is found to be little variation in the required amount of masonry for central angles ranging between 120° and 145°.

The relative positions of the various horizontal sections of a constant-angle dam are shown in the plan view in Fig. 23. Here, six levels are considered. At each such level are indicated the center of the curves for the faces of the dam at that level, the radius of the down-stream face, the thickness of the dam, and the contour lines, or the lines through points in the valley having the same elevation as the level under consideration. The con-

tour lines indicate the general slopes of the valley sides, the slopes being steep where the contour lines are close together and flat where they are far apart. For the lowest level considered, the center of curvature is at a and the radius of the down-stream face is R_1, the thickness of the concrete is t_1, and the contour lines along the valley at that level are the lines marked I. Similarly for the next level, the center of curvature is at b, the radius of the down-stream face is R_2, the thickness is t_2, and the contour lines are marked II.

The cross-section of a constant-angle dam is shown in Fig. 24. The radii of curvature at various heights above the foundation are given in the table accompanying the illustration, and the thickness of the concrete at each of these heights is marked on the section.

Height of Dam above River Bed	Up Stream Radius
168'	333'
156'	333'
144'	325'
132'	316'
120'	305'
108'	294'
96'	282'
84'	270'
72'	257.5'
60'	243.5'
48'	226.5'
36'	207'
24'	187.5'
12'	168'
0'	147.5'

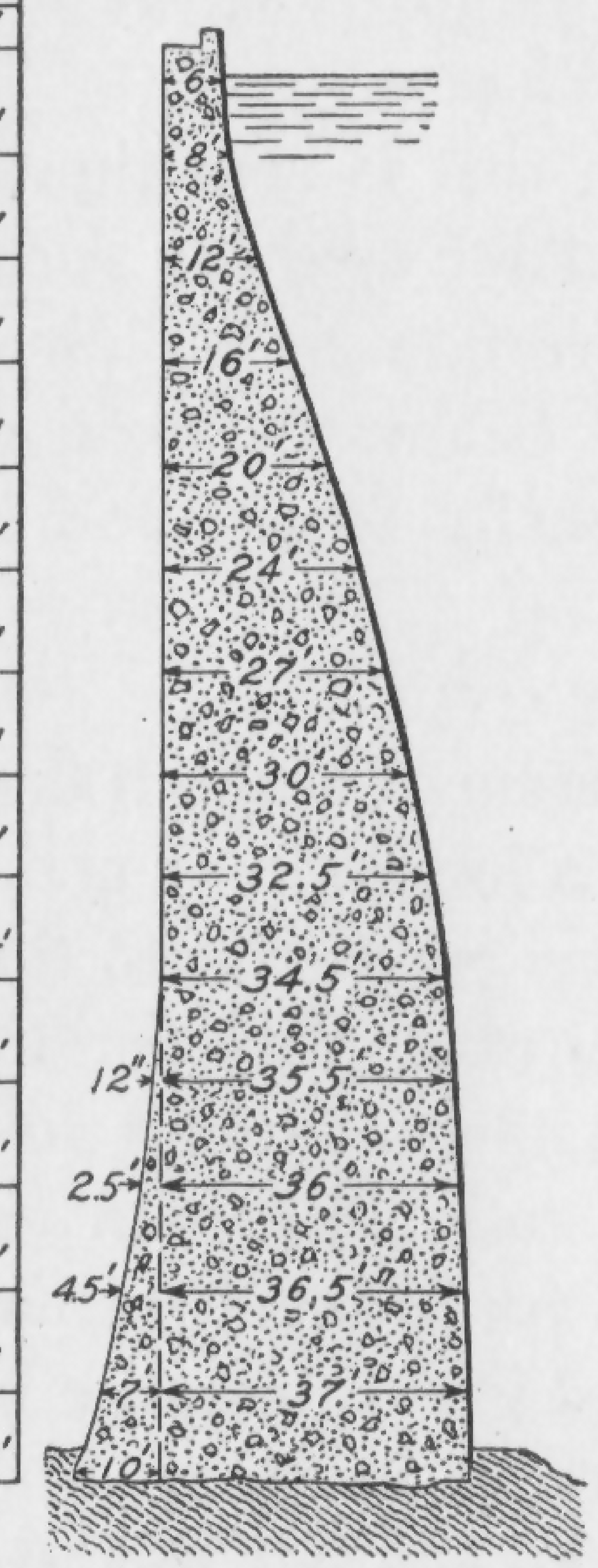

Fig. 24

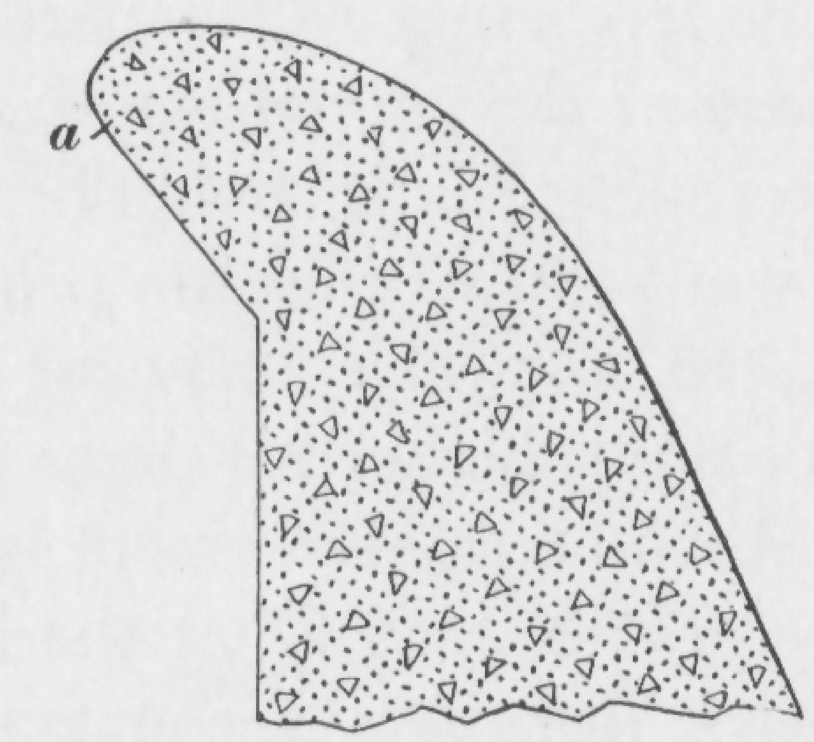

Fig. 25

65. Spillway Section for Arch Dam.—When a constant-angle arch dam is designed only for stability against horizontal thrust, it is usually found economical to make the down-stream face vertical for practically the entire height of the dam, as shown in Fig. 24. But, such a design is not suitable for a spillway section where the volume of overflow is large, because the falling water would not be guided properly. If it is feasible to locate the spillway for an arch dam in a gorge beyond one end of the dam, it is generally preferable to adopt that location for

the spillway and to design the dam itself as a non-overflow section. However, in many cases it is necessary to build part of the dam as a spillway section. Trembling of the dam and erosion of the foundation bed are then usually prevented by shaping the down-stream face of the overflow section as explained for a spillway section of a gravity dam. The up-stream face is generally made vertical for this portion of the dam and a varying radius for the down-stream face at different levels gives that face the proper shape. Where the dam is quite thin at the top, a suitable curve for the crest may be provided by means of an overhanging lip, as shown in Fig. 25 at *a*.

Another method of construction that is sometimes employed is to make the down-stream face of the spillway vertical and to provide a water cushion for the overflow by building an auxiliary gravity dam a short distance below the main dam. The height of the auxiliary dam is usually from one-fifth to one-quarter of that of the main dam.

66. Details of Design.—Each strip of an arch dam between any two levels may be compared to a long column and, therefore, the ratio of the curved length of any such strip to the thickness of the concrete in it should be given some consideration. Since the lower parts of the dam provide lateral support for the upper parts, the allowable value of the ratio of length to thickness may be much higher for dams than for ordinary columns. However, when no reinforcement is used in the dam, the ratio should not be greater than 75 at the top or 25 at the mid-height; conservative designers recommend 60 and 20 for the limiting values.

Vertical reinforcement is sometimes provided in the upper part of an arch dam to distribute ice thrust and also to stiffen that portion of the structure when the ratio of the curved length to the thickness of the concrete is high. However, continuous horizontal reinforcement cannot be used to advantage in the dam, because the steel rods cannot be properly anchored in the abutments and because it is not desirable to have the rods pass through the vertical construction joints, which are placed so as to localize the cracks in the concrete in radial planes.

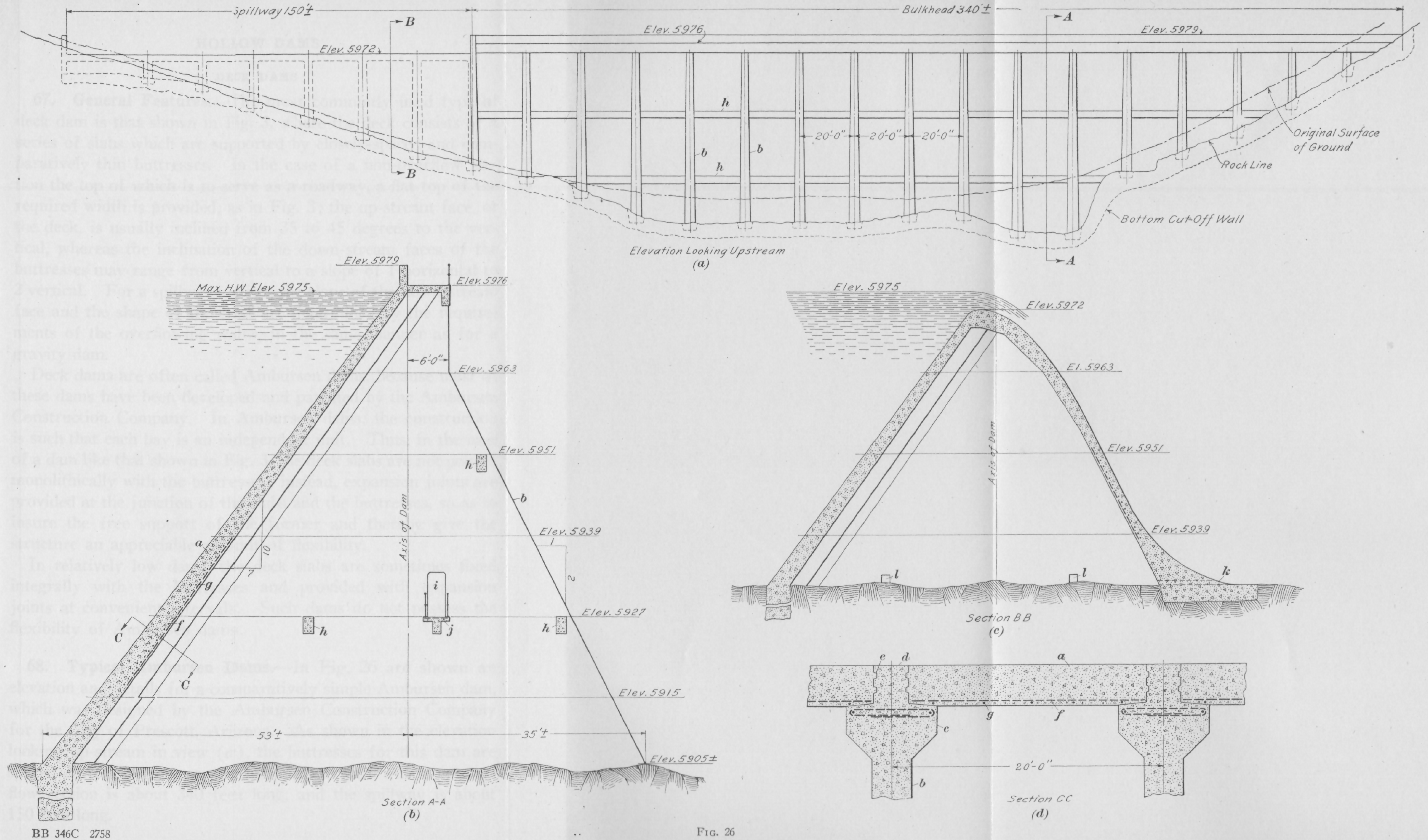

FIG. 26

HOLLOW DAMS

DECK DAMS

67. General Features.—The most commonly used type of deck dam is that shown in Fig. 3, where the deck consists of a series of slabs which are supported by closely-spaced and comparatively thin buttresses. In the case of a non-overflow section the top of which is to serve as a roadway, a flat top of the required width is provided, as in Fig. 3; the up-stream face, or the deck, is usually inclined from 35 to 45 degrees to the vertical, whereas the inclination of the down-stream faces of the buttresses may range from vertical to a slope of 1 horizontal to 2 vertical. For a spillway section, the slope of the down-stream face and the shape of the crest are determined by the requirements of the overflowing water, in the same manner as for a gravity dam.

Deck dams are often called Ambursen dams, because most of these dams have been developed and patented by the Ambursen Construction Company. In Ambursen dams, the construction is such that each bay is an independent unit. Thus, in the case of a dam like that shown in Fig. 3, the deck slabs are not poured monolithically with the buttresses; instead, expansion joints are provided at the junction of the slabs and the buttresses, so as to insure the free support of the former and thereby give the structure an appreciable amount of flexibility.

In relatively low dams, the deck slabs are sometimes fixed integrally with the buttresses and provided with expansion joints at convenient intervals. Such dams do not possess the flexibility of Ambursen dams.

68. Typical Ambursen Dams.—In Fig. 26 are shown an elevation and details for a comparatively simple Ambursen dam, which was designed by the Ambursen Construction Company for the City of Prescott, Arizona. As shown in the elevation looking up-stream in view (*a*), the buttresses for this dam are spaced 20 feet on centers, the bulkhead or part with non-overflow section is about 340 feet long, and the spillway is about 150 feet long.

A typical cross-section through the bulkhead at *A-A* is shown in view (*b*) and one through the spillway at *B-B* is shown in view (*c*). The manner in which the deck slabs *a* are supported by the buttresses *b* is shown in view (*d*), which is the cross-section taken at *C-C* at right angles to the deck and buttresses. The deck slabs rest on corbels *c* projecting from the buttresses. The part *d* of the buttress that extends between adjacent slabs is grooved, as at *e*, and a heavy coating of asphalt putty is applied to the surfaces of the buttress that come in contact with the slab, thus forming water-tight expansion joints. Since the deck slabs are freely supported on the buttresses, the structure is not continuous and each bay or unit of the dam is independently stable in itself. Also, the reinforcing bars *f* and *g* in the water-bearing members are then near the under side of those members and, therefore, are less liable to rust because of coming in contact with the water in the reservoir if the concrete protection should crack. In order to provide additional resistance to lateral forces, reinforced struts, *h* in views (*a*) and (*b*), are run between the buttresses. A continuous passageway through the structure is usually provided by leaving openings in the buttresses, as at *i*, and building a reinforced-concrete walkway *j* between them.

Since the dam in this case is comparatively low, the struts and walkway between the buttresses were omitted in the spillway, as shown in view (*c*); but for higher dams it is customary to employ them between the buttresses of the spillway in the same manner as between those of the bulkhead. Provision is usually made for draining the interior of the dam at the spillway by means of openings *k* located at intervals in the bucket. In dams where these openings are very large, considerable air from the interior of the dam may be carried away by the water flowing over the bucket. Hence, in order to prevent the formation of a partial vacuum in the interior of the dam and thereby increase the effect of the water pressure on the faces of the dam, it is often necessary to provide large openings in the buttresses in addition to those for the walkway. In the spillway section in view (*c*), the openings *l* for drainage between buttresses also help admit air.

69. If conditions require the construction of a deck dam on a compressible soil, a comparatively low pressure on the foundation bed may be obtained by supporting the buttresses on a continuous reinforced-concrete mat. A section through the floor of an Ambursen dam resting on a relatively weak soil is shown in Fig. 27, where the buttresses *a* are supported on the mat *b*. In order to safeguard against leakage of water under the floor, adequate cut-off walls should be provided at the up-stream and down-stream ends of the floor mat. Further protection against uplift should be made by providing an ample

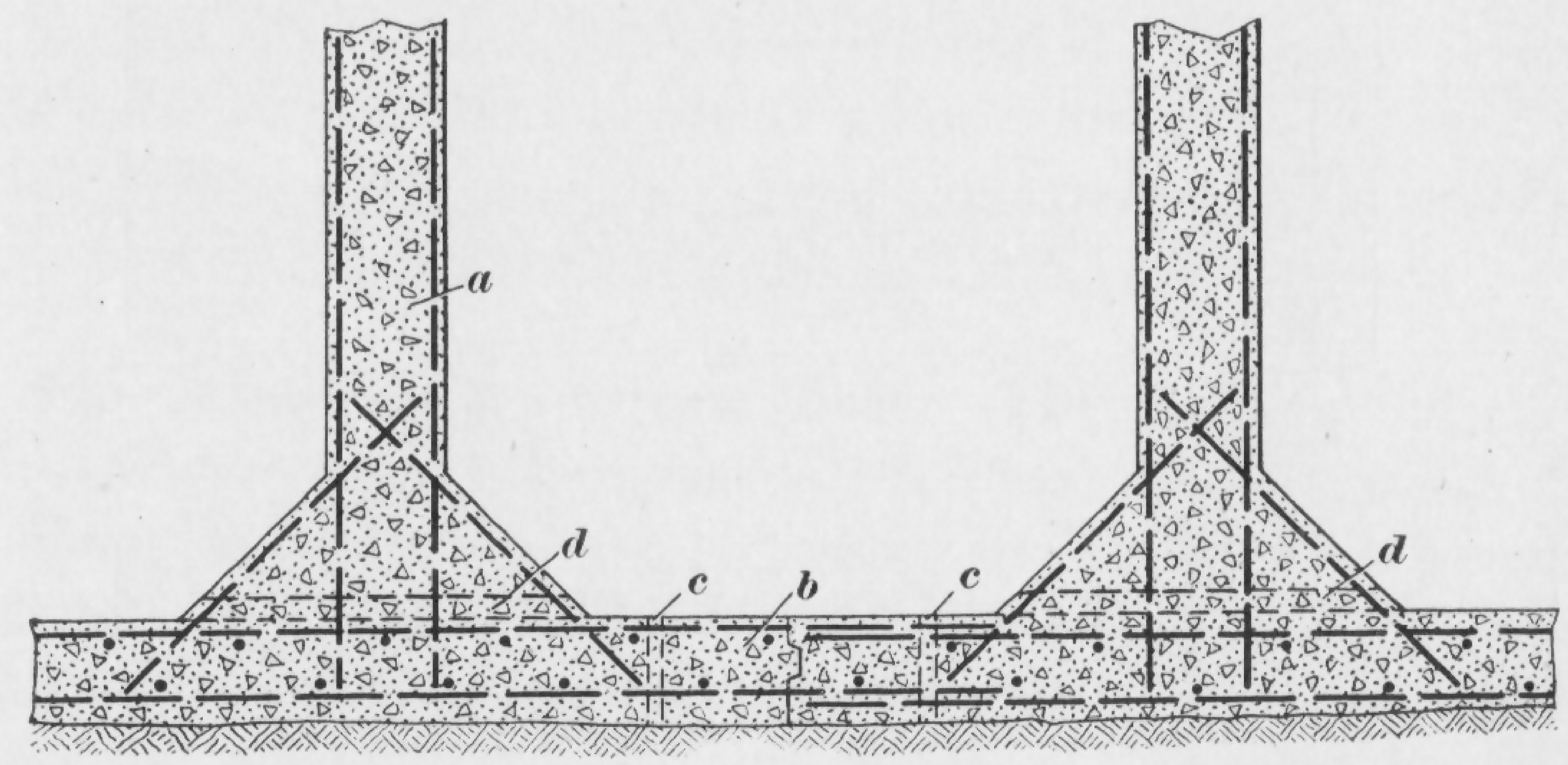

FIG. 27

number of weep holes *c* through the mat and lateral drains under the floor. In addition, the openings *d* insure drainage between buttresses.

70. The type of construction shown in Fig. 26 has been used successfully not only for low dams but also for fairly high dams. The buttresses are comparatively thin reinforced-concrete walls, usually spaced from 16 to 30 feet on centers. However, for dams that are over 150 feet high, economical results have been obtained by using relatively massive buttresses spaced from 40 to 60 feet on centers. As shown in the horizontal section in Fig. 28 (*a*), the buttresses are enlarged at their up-stream and down-stream ends in order to shorten the spans of the deck slabs and to increase the stability of the buttresses.

Another type of deck that has been found advantageous for dams of various heights is the so-called *round-head buttress type,*

shown in horizontal section in view (*b*). For dams of moderate height the spacing of buttresses may be as low as 30 feet on centers, whereas for higher dams it may range from 40 to 70 feet. In this type of dam, which was developed by Fred A. Noetzli, the up-stream end of the buttress is flared out to form an enlarged round head which meets the heads of adjacent buttresses so that no deck slabs are required. The special shape of the head causes the pressure of the water to be transmitted radially to

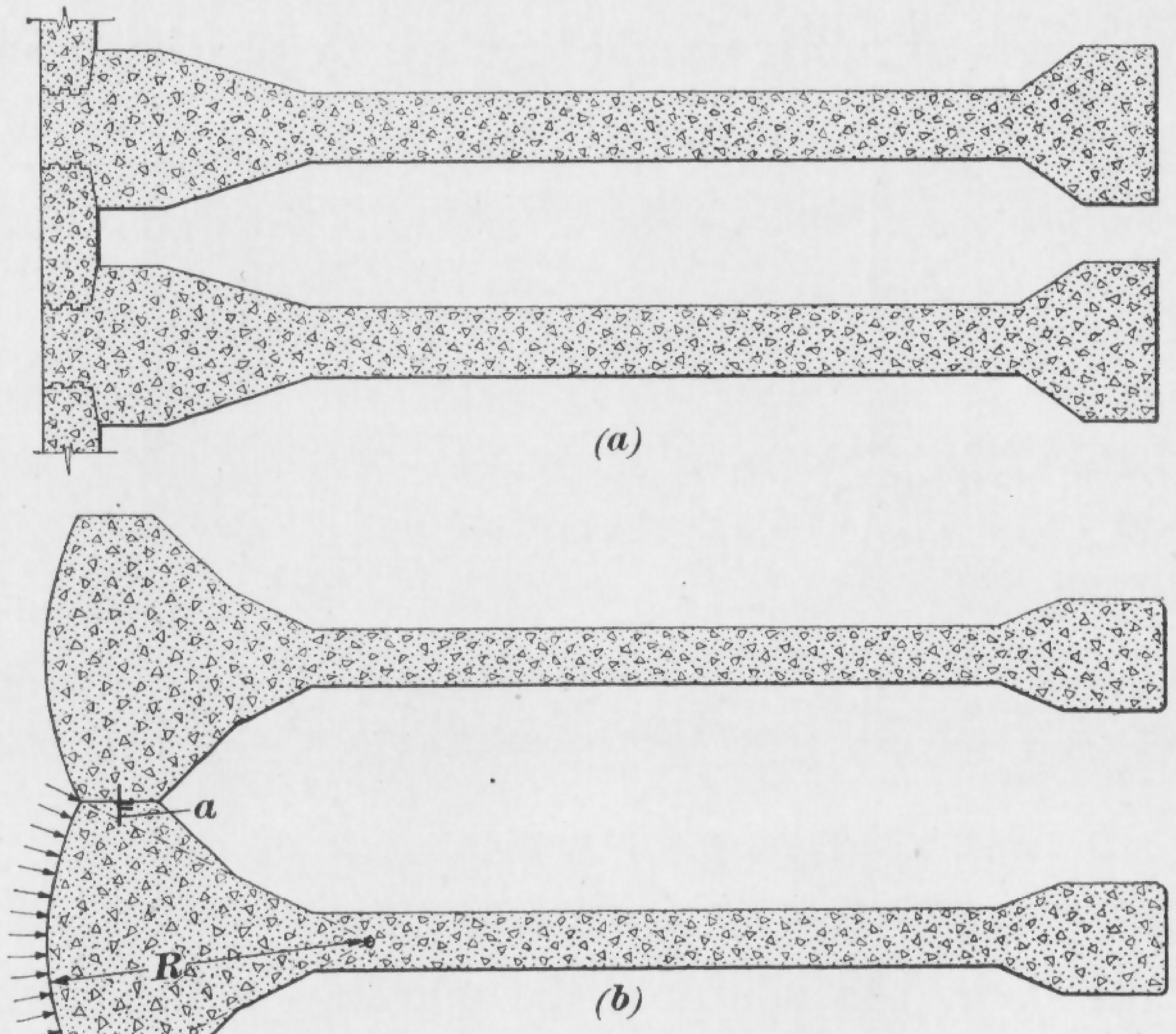

FIG. 28

the buttress, and the bending or diagonal-tension stresses in the water-bearing member are practically negligible. Therefore, the only reinforcement that need be used in this case is for shrinkage or temperature requirements. At the junction of the buttress heads, construction joints are provided to insure flexibility; also, copper flashing is inserted, as at *a*, to protect the joint against possible leakage. Various other types of deck dams have been developed.

71. Loads on Deck Dams.—Since the up-stream face of a deck dam is inclined, the vertical component of the water pressure on the deck helps resist the tendency of the horizontal

component of that pressure to overturn or slide the dam along its base or any horizontal plane. In fact, for the slopes generally used, the resultant of all forces acting above any horizontal section through a buttress comes near the center of that section. No uplift need be considered because usually the bearing areas of the buttresses on the foundation bed are comparatively small and whatever water seeps under the deck can readily escape. Also, when the buttresses are supported on a floor mat, the weep holes in the mat and the drains under the floor remove the seepage so that the uplift pressure may be disregarded. The sloping deck tends to eliminate practically all ice pressure against it because, as the water at the surface of the reservoir freezes, the expansion not only produces a horizontal thrust against the deck slab, but also causes an upward movement of the ice layer, which breaks the contact between that layer and the slab.

72. Design of Up-Stream Deck.—The up-stream deck of an Ambursen dam is designed as a reinforced-concrete slab that is simply supported by the buttresses and sustains the pressure of the water. Since this pressure increases as the depth below the surface of the water becomes greater, the strength of the deck slab must also increase with the depth. Usually, the thickness of the slab is increased gradually, as in Fig. 26 (*b*), and the amount of reinforcement is increased at intervals. In this case, the main reinforcement consists of straight rods, *f* in view (*d*), running between the faces of the abutments. To the under side of these rods are wired the secondary rods *g*.

In designing the deck slab, it is convenient to consider strips 1 foot wide at various levels and to determine the required thickness of concrete and amount of reinforcement in the strip at each level. The thickness of the slab is changed gradually, but it is not practical to vary the amount of reinforcement continually. Therefore the required size and spacing of the rods at any level is maintained uniformly from that level to the next one above it. The load on the slab is the sum of the water pressure acting normal to the slab and the component of the weight of the slab that is perpendicular to the face of the slab. Since the slab is simply supported, the span may be taken as the

distance between centers of the supporting corbels on the buttresses; for dams of low or moderate height with comparatively thin buttresses, the span may be assumed as 1 foot less than the distance between the centers of buttresses.

73. In the case of a hollow dam, conservative values should be used for the allowable unit stresses in the concrete. When the thickness of the slab is determined by the bending moment, and the actual shearing unit stress in the slab at the buttress is

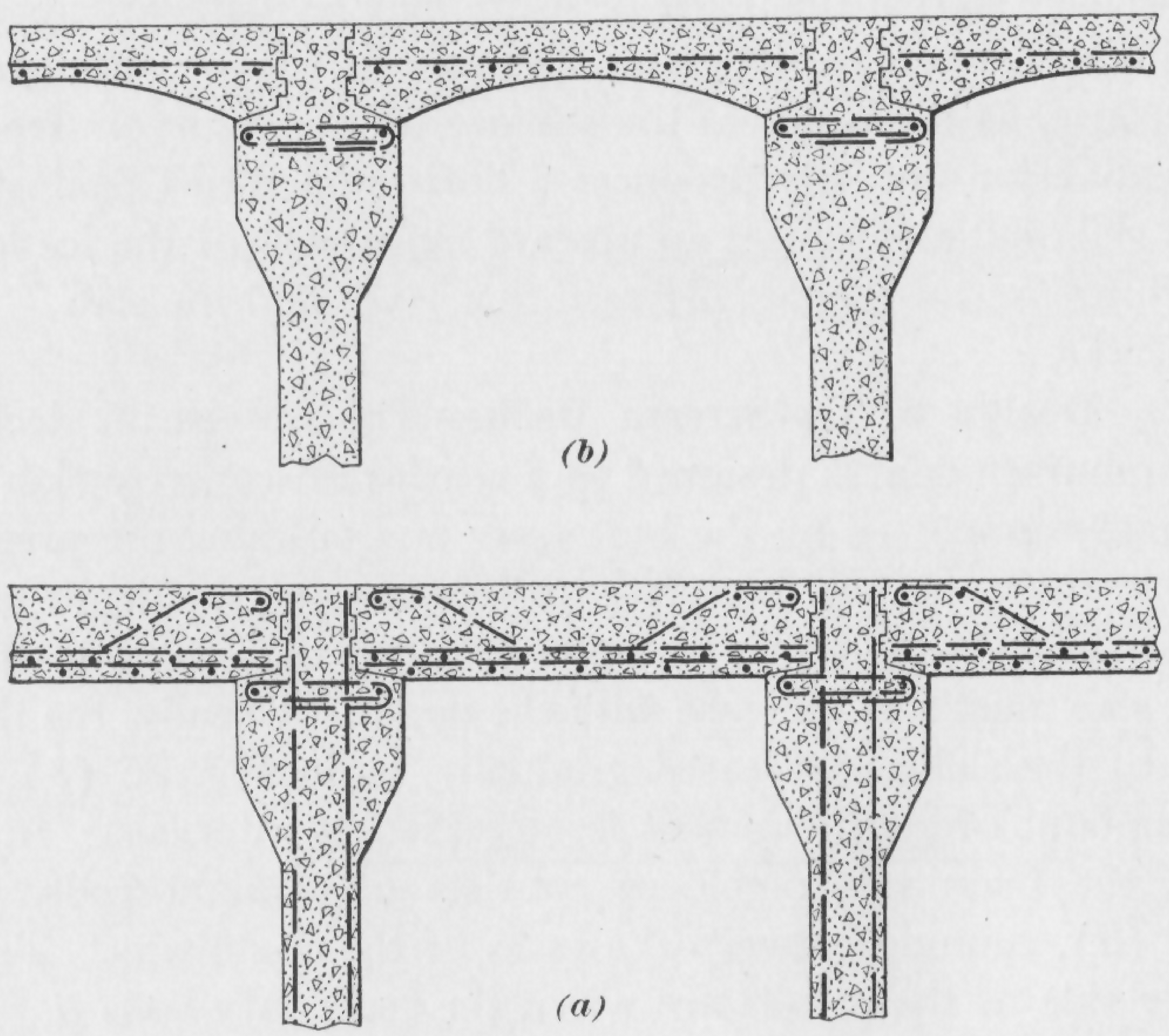

FIG. 29

not much greater than the allowable value for the concrete, web reinforcement may be provided by bending up some of the main tension bars as in Fig. 29 (*a*). Sometimes, the shearing unit stress is reduced within safe limits by increasing the thickness of the slab near the buttress. The under surface of the slab is then arched, as shown in view (*b*), in order to save concrete, but the reinforcement is straight and parallel to the upper surface of the slab. Since all the additional concrete is then below the steel, it is not effective in increasing the flexural strength of the slab. Furthermore, the arch action is not effective while

the steel reinforcement is intact. However, in case the reinforcement should be ruptured, the arching would help resist the complete failure of the slab. In determining the depth required to resist the shear, the slab is treated as a plain-concrete beam. The secondary reinforcement g in Fig. 26 (*d*) usually consists of $\frac{1}{2}$-inch square or $\frac{5}{8}$-inch round bars spaced about 24 inches on centers and tied to the main bars. In some dams the deck consists of a combination of flat slabs and arches that can resist the water pressure by arch action alone.

EXAMPLE.—The up-stream deck of an Ambursen dam of non-overflow section is inclined at an angle of 45° to the vertical, and the buttresses are placed 16 feet on centers. Find the required total thickness of the slab and the size and spacing of the main reinforcement at a depth of 30 feet below the water surface, if $f_c=650$, $f_s=16{,}000$, and $v_c=40$ pounds per square inch, and $n=15$.

SOLUTION.—The total load on a strip of the slab 1 ft. wide is determined as follows: The normal water pressure is $62.5\times30=1{,}875$ lb. per sq. ft. If the component of the weight of the deck at right angles to its face is taken as 15 per cent. of the water pressure, or, say, 280 lb. per sq. ft., the total load per sq. ft. is $1{,}875+280=2{,}155$ lb. The span may be assumed as $16-1=15$ ft., and the bending moment is

$$M'=\frac{2{,}155\times15^2}{8}=60{,}600 \text{ ft.-lb.}$$

For the given values of f_s, f_c, and n, the coefficient K is 107.7 and the economic steel ratio $p_e=.0077$. Hence, the required effective thickness of the slab is

$$d=\sqrt{\frac{M'}{K}}=\sqrt{\frac{60{,}600}{107.7}}=23.7, \text{ say 24 in.}$$

If 4 in. of concrete is allowed outside the steel, the total thickness of the slab will be 28 in. and its weight per sq. ft. will be $\frac{28}{12}\times150=350$ lb. The component of this weight that is perpendicular to the face of the slab is $350 \sin 45°=248$ lb., which is close enough to the assumed weight of 280 lb. when the total load is so great.

The corrected load per sq. ft. is $1{,}875+248=2{,}120$ lb., the maximum shear in the slab is

$$V=\frac{2{,}120\times15}{2}=15{,}900 \text{ lb.}$$

and the maximum bending moment is

$$M' = \frac{2{,}120 \times 15^2}{8} = 59{,}600 \text{ ft.-lb.}$$

The shearing unit stress is

$$v = \frac{V}{bjd} = \frac{15{,}900}{12 \times .874 \times 24} = 63.2 \text{ lb. per sq. in.}$$

The necessary web reinforcement to resist diagonal tension can be provided by bending up some of the main tension rods. However, if desired, the slab can be made thicker at the buttresses to reduce the shearing stress to 40 lb. per sq. in. If the weight of the additional concrete is disregarded and the slab is treated as a rectangular beam of plain concrete, the required total thickness at each buttress is

$$d' = \frac{3}{2}\frac{V}{vb} = \frac{3}{2} \times \frac{15{,}900}{40 \times 12} = 49.7, \text{ say } 50 \text{ in.}$$

The required area of the main reinforcement per in. of width at the center of the span is

$$A_s = \frac{M'}{f_s jd} = \frac{59{,}600}{16{,}000 \times .874 \times 24} = .178 \text{ sq. in.}$$

This area can be provided by 1-in. round bars spaced $.785 \div .178 = 4.41$, say $4\frac{1}{4}$ in. apart. The unit bond stress is then

$$u = \frac{V}{jdO} = \frac{15{,}900}{.874 \times 24 \times \frac{12}{4.25} \times 3.14} = 85.5 \text{ lb. per sq. in.,}$$

which is below the usual allowable value of 100 lb. per sq. in. for deformed bars. Hence, the total thickness could be made 28 in. and the main reinforcement could consist of 1-in. round deformed bars spaced $4\frac{1}{4}$ in. apart. Ans.

74. Design of Buttresses.—Each buttress is designed to resist the forces acting on a length of dam equal to the distance between centers of buttresses. These forces are the weight of the concrete in the deck and buttress and the water pressure on the up-stream face. In carrying out the computations, it is found convenient to resolve the water pressure into its horizontal and vertical components, each applied at the center of pressure. For the reasons previously given, neither uplift nor ice pressure need be considered in the design. In the case of a spillway section, the weight of the water on the down-stream face is usually neglected. Also, in determining the compressive stresses at the base of a buttress, it is customary to disregard the horizontal area of the deck slab and to consider only the bearing area of the buttress itself.

The forces that act on a buttress tend to overturn it, slide it, or crush the concrete in it. Consequently, each buttress should be designed so as to meet the following requirements: (1) The resultant of all the forces acting on the buttress should cut the base within the middle third; when the up-stream face has an inclination of about 45°, the resultant for both full and empty reservoir generally passes very close to the center of the base. (2) The relation between the horizontal and vertical components of the resultant force must be such that there will be no danger of the dam's sliding. (3) The maximum compressive unit stress in the concrete must be within safe limits; since the amount of the resultant force is much greater for a full reservoir than for an empty reservoir, and the eccentricity is not great for either condition, the stress need only be calculated for a full reservoir. For comparatively low dams, the design may be based on allowable maximum vertical unit stresses that are assumed sufficiently low, but for dams of moderate or great height an investigation of the inclined stresses is essential. Care should be taken to avoid tensile stresses in the up-stream ends of the buttresses.

75. The usual procedure in investigating the buttress at any level is somewhat similar to that described for a section of a gravity dam. However, the design cannot be carried out by proceeding from level to level. It is customary to select at the start tentative dimensions for the entire buttress and then to investigate the horizontal sections at various levels and to make whatever adjustments may be desirable. The thickness of a buttress at the top should not be less than 12 inches for a non-overflow section and 15 inches for a spillway section. The increase in thickness may be made either gradually, by sloping the sides of the buttress as in Fig. 26 (*a*), or in steps at vertical intervals, as in Fig. 30.

In order to stiffen the buttresses laterally, reinforced-concrete struts, *e* in Fig. 3 and *h* in Fig. 26 (*a*) and (*b*), are run between

them. The reinforcement in the struts is usually made continuous for at least three bays; in some comparatively short dams it is made continuous throughout the entire length of the dam. When the buttresses are stressed fully in compression, the horizontal or vertical spacing of the struts should not exceed twelve times the thickness of the buttresses. Sometimes the buttresses are further stiffened by embedding in them horizontal rods that extend between the struts. Shrinkage rods are usually provided near both faces of the buttresses. In the Stony Gorge Dam built in California by the United States Bureau of Reclamation, it was found necessary to insert such rods running horizontally, vertically, and parallel to the deck slab.

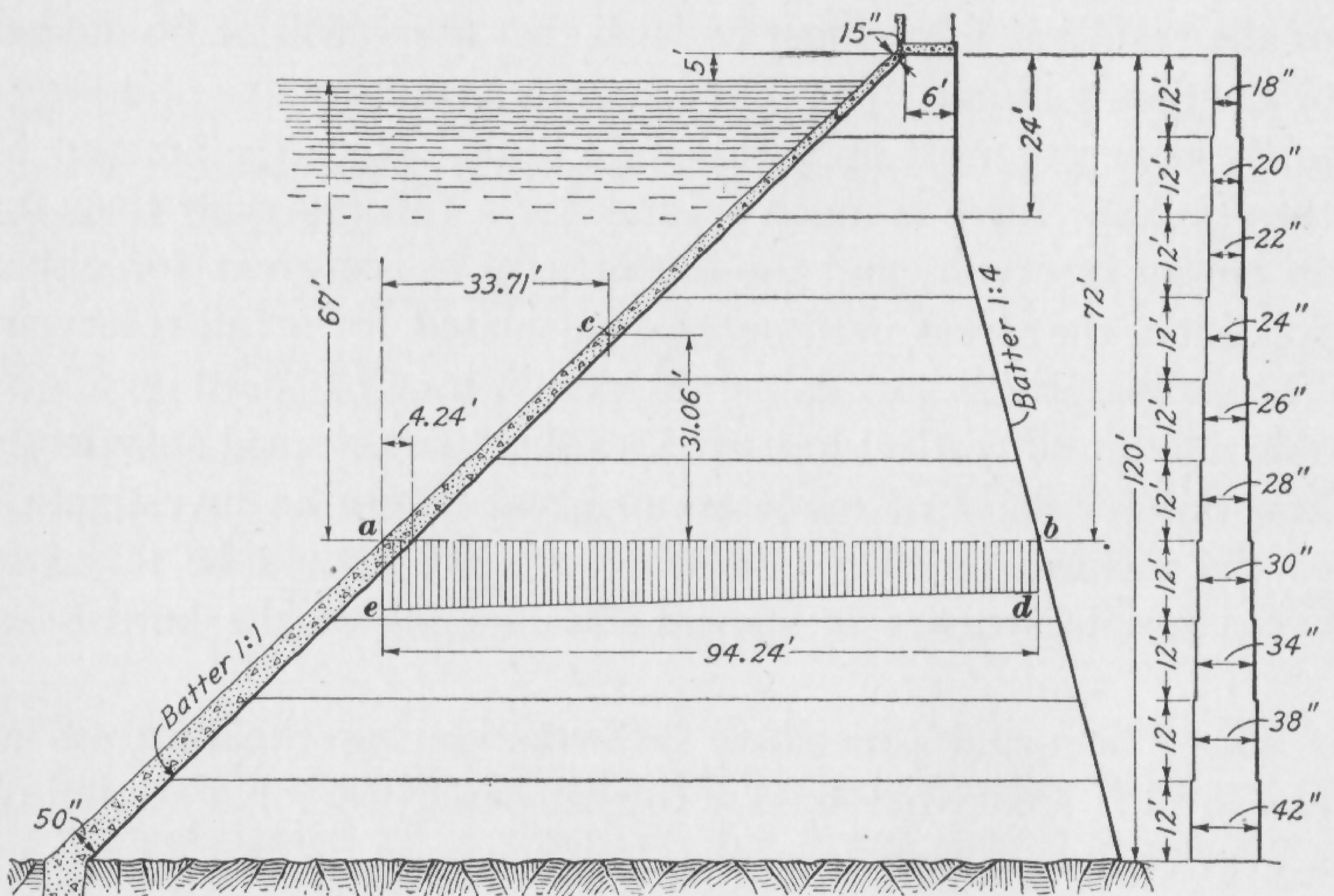

Fig. 30

Example.—In Fig. 30 are shown the dimensions of the up-stream slab of an Ambursen dam and the tentative dimensions of a buttress. The thickness of the slab increases uniformly at the rate of $3\frac{1}{2}$ inches in each 12 feet of vertical distance from 15 inches at the top to 50 inches at a depth of 120 feet below the top. The thickness of the buttress increases from 18 to 42 inches in steps at vertical intervals of 12 feet, and the thickness of the top slab is 12 inches. The buttresses are spaced 18 feet on centers. (*a*) Assuming the coefficient of friction to be .65, investigate the stability of the dam against sliding at the horizontal section *ab*, which is 67 feet below the water level. (*b*) Determine the maximum and minimum vertical compressive unit stresses in the buttress at that level.

Solution.—(*a*) In investigating the stability against sliding, the first step is to determine the amounts of the vertical and horizontal forces acting above the level *ab*, a length of 18 ft. of dam being considered. The calculations for the volume of the concrete and its moment about the point *a* are shown in the accompanying tabular form; the moments are required in locating the line of action of the resultant force which is needed in part (*b*). Since the thickness of the deck slab varies at the rate of $3\frac{1}{2}$ in. in 12 ft., the thickness at *a* is $15+\frac{72}{12}\times 3.5=36$ in. The arm of the portion of the deck slab under consideration may be found as follows: The vertical distance from the level *ab* to the center of gravity *c* of that portion is $\frac{72}{3}\times\frac{36+2\times 15}{36+15}=31.06$ ft., and the thickness of the slab at the center of gravity *c* is $36-\frac{31.06}{12}\times 3.5=26.93$, or say, 27 in. Also, the horizontal projection of the thickness of the slab at the level *ab* may be taken as $3\div\sin 45°=4.24$ ft., and at the level through the center of gravity, as $2.25\div\sin 45°=3.18$ ft. Hence, the required arm is $4.24+31.06\tan 45°-\frac{1}{2}\times 3.18=33.71$ ft.

The total weight of the concrete is $W=10{,}850\times 150=1{,}627{,}500$ lb. Also, the horizontal component of the water pressure on the deck slab is

$$T=31.25\times 67^2\times 18=2{,}525{,}000 \text{ lb.}$$

To determine the vertical component of the water pressure on the deck, it is necessary to find first the horizontal projection of the exposed surface. The difference between the thickness of the slab at *a* and that at the water surface is $\frac{67}{12}\times 3\frac{1}{2}=19.54$ in. or 1.63 ft. and the horizontal projection of that difference is $\frac{1.63}{\sin 45°}=2.3$ ft. Hence, the required projection of the face of the slab is $67+2.3=69.3$ ft. and the vertical component of the water pressure is

$$V=62.5\times 69.3\times 18\times\frac{67}{2}=2{,}611{,}700 \text{ lb.}$$

The total vertical force is $W+V=1{,}627{,}500+2{,}611{,}700=4{,}239{,}200$ lb. and the resistance to sliding is $4{,}239{,}200\times .65=2{,}755{,}500$ lb., which is somewhat greater than the horizontal thrust of 2,525,000 lb. Therefore, the section may be considered safe as far as sliding is concerned.

(*b*) In determining the vertical compressive unit stresses, the first step is to find the eccentricity of the resultant force on the section *ab*. The moment of the weight of the masonry about the point *a* is $536{,}300\times 150=80{,}450{,}000$ ft.-lb.; that of the horizontal component of the water

Part	Volume, in Cu. Ft.		Arm, in Ft.	Moment
Top slab	$6\times18\times1$	$=108$	79.24	8,560
Deck slab	$\dfrac{72}{\sin 45^\circ}\times18\times\left(\dfrac{15+36}{2\times12}\right)$	$=3,895$	33.71	131,300
Buttress:				
levels 0–12	$6\times12\times\frac{18}{12}$	$=108$	79.24	8,560
	$\frac{1}{2}\times12\times12\times\frac{18}{12}$	$=108$	72.24	7,800
levels 12–24	$18\times12\times\frac{20}{12}$	$=360$	73.24	26,370
	$\frac{1}{2}\times12\times12\times\frac{20}{12}$	$=120$	60.24	7,230
levels 24–36	$30\times12\times\frac{22}{12}$	$=660$	67.24	44,380
	$\frac{1}{2}\times12\times12\times\frac{22}{12}$	$=132$	48.24	6,370
	$\frac{1}{2}\times3\times12\times\frac{22}{12}$	$=33$	83.24	2,750
levels 36–48	$45\times12\times\frac{24}{12}$	$=1,080$	62.74	67,760
	$\frac{1}{2}\times12\times12\times\frac{24}{12}$	$=144$	36.24	5,220
	$\frac{1}{2}\times3\times12\times\frac{24}{12}$	$=36$	86.24	3,100
levels 48–60	$60\times12\times\frac{26}{12}$	$=1,560$	58.24	90,850
	$\frac{1}{2}\times12\times12\times\frac{26}{12}$	$=156$	24.24	3,780
	$\frac{1}{2}\times3\times12\times\frac{26}{12}$	$=39$	89.24	3,480
levels 60–72	$75\times12\times\frac{28}{12}$	$=2,100$	53.74	112,850
	$\frac{1}{2}\times12\times12\times\frac{28}{12}$	$=168$	12.24	2,060
	$\frac{1}{2}\times3\times12\times\frac{28}{12}$	$=42$	92.24	3,870
		10,849		536,290

pressure is $2,525,000\times\frac{67}{3}=56,390,000$ ft.-lb.; and that of the vertical component is $2,611,700\times\frac{69.3}{3}=60,300,000$ ft.-lb. Hence, the moment of the resultant force is $80,450,000+56,390,000+60,330,000=197,170,000$ ft.-lb. and the distance from a to its point of application on ab is $197,170,000\div4,239,200=46.51$ ft.

Since the horizontal projection of the thickness of the deck slab at a is 4.24 ft., the total width of ab is $4.24+72+6+12=94.24$ ft. and the distance from a to the center of ab is 47.12 ft. Hence, the eccentricity of the resultant force is $47.12-46.51=.61$ ft., and the maximum vertical unit pressure at the point a is

$$p=\frac{R_v}{b}\left(1+\frac{6e}{b}\right)=\frac{4,239,200}{94.24}\times\left(1+\frac{6\times.61}{94.24}\right)=46,730 \text{ lb. per sq. ft.}\quad \text{Ans.}$$

Also, the minimum vertical unit pressure at b is

$$p_1=\frac{4,239,200}{94.24}\times\left(1-\frac{6\times.61}{94.24}\right)=43,240 \text{ lb. per sq. ft.}\quad \text{Ans.}$$

The variation in the vertical unit pressure on the section ab may be represented by the trapezoid $abde$, in which ae is 46,730 and bd is 43,240 lb. per sq. ft.

76. Design of Spillway Section.—The up-stream slab of a spillway dam of the Ambursen type is designed in the same way as the slab of a non-overflow dam. Also, the shape of the crest and bucket is based on the principles given for a spillway section of a gravity dam. However, since the up-stream face of a hollow dam slopes at an angle of about 45°, the curve at the crest corresponding to ef in Fig. 19 may be modified. Thus the horizontal distance between e and f is usually made about $.2\,H$ for a hollow dam instead of $.25\,H$ for a gravity section, and the vertical distance is made $.05\,H$ instead of $.11\,H$.

The thickness of the concrete and the amount of reinforcement at the crest and along the down-stream face of the spillway cannot be calculated accurately on account of the indeterminate loading. However, the thickness is always made greater at the crest than in the sloping portion, as indicated in Fig. 26 (c). Also, it is convenient to have the size and spacing of the reinforcing bars uniform throughout the entire down-stream slab. The thickness at the discharge end of the bucket must be ample to withstand shocks.

EXAMPLE.—The deck slab of a spillway portion of an Ambursen dam is supported by buttresses that are spaced 20 feet on centers. If the height of the water above the crest is 8 feet, what is the maximum bending moment due to the water pressure on a strip of slab 1 foot wide that is at a vertical distance of 24 feet below the crest?

SOLUTION.—The depth from the water surface to the strip under consideration is $8+24=32$ ft. and, by formula 1, Art. 20, the normal unit pressure on the strip is

$$p=62.5\,h_x=62.5\times 32=2{,}000 \text{ lb. per sq. ft.}$$

Since the span of the slab may be taken as $20-1=19$ ft., the required bending moment, not including that due to the weight of the deck, is

$$M'=\frac{2{,}000\times 19^2}{8}=90{,}250 \text{ ft.-lb. Ans.}$$

77. Practical Considerations.—In the case of a hollow dam the dimension that determines to a large extent the economy of the structure is the distance between buttresses. In order to obtain the best spacing for a particular dam, it is usually advisable to work out complete designs for two or more spacings. For a given distance between buttresses, the usual procedure is to design the parts in the following order: the up-stream slab; the buttresses; the footings for the buttresses, if they are required; the crest, down-stream slab, and bucket for the spillway section; and the struts running between the buttresses.

EXAMPLES FOR PRACTICE

1. The up-stream face of an Ambursen dam of the non-overflow type makes an angle of 45° with the vertical. If the buttresses are spaced 15 feet on centers, the allowable unit stresses are $f_c=650$, $f_s=16{,}000$, and $v_c=40$ pounds per square inch, and $n=15$, determine (*a*) the required total thickness of the slab to resist the bending moment at a depth of 42 feet below the water surface, allowing 4 inches of concrete outside the center of the reinforcement, and (*b*) the spacing of 1-inch round deformed rods for the main reinforcement at that depth.

Ans. $\begin{cases}(a)\ 30 \text{ in.}\\(b)\ 4 \text{ in.}\end{cases}$

2. If the shearing unit stresses are to be kept below the allowable value by increasing the thickness of the slab at the buttresses, what is the required thickness to resist the maximum shear at the level investigated in the preceding example? Ans. 64 in.

3. For the dimensions shown in Fig. 30 and described in the example of Art. 75, determine the factor of safety against sliding at the horizontal section 91 feet below the water level. Ans. 1.08

4. Calculate the maximum vertical compressive unit stress in the buttress at the section considered in example 3.

Ans. 63,000 lb. per sq. ft.

MULTIPLE-ARCH DAMS

78. General Features.—Where good foundation material is available for the support of the buttresses of a hollow dam, it may be best to provide a multiple-arch dam, in which the deck consists of a series of arches spanning between the buttresses. The details of construction of a dam of this type are shown in Fig. 31. In view (*a*) is a vertical section midway between buttresses and an elevation of a buttress, in (*b*) is the vertical cross-section *A-A* through the buttress, and in (*c*) is the enlarged cross-section *B-B*. An economical design for high multiple-arch dams is obtained when the distance between centers of buttresses is 60 to 75 feet. For comparatively low dams, a span of about 30 to 40 feet is usually preferable.

In order to insure safety against sliding on the foundation bed, multiple-arch dams should be built on solid rock and the arches and buttresses should be well keyed into the rock. If the dam rests on material other than solid rock, ample resistance to sliding must be provided and the bearing area must be sufficient to prevent irregular settlement of the buttresses.

79. Details of Deck Slab.—The arches forming the deck are parts of cylinders, but the axes of these cylinders are inclined to the vertical. Therefore, a cross-section through an arch taken at right angles to the axis is inclined to the horizontal and, since the different parts of the section are at different depths below the water surface, they are subjected to different unit pressures. This non-uniformity in loading is an important consideration near the top of the dam, but is of little consequence near the bottom. The load on an arch also includes the component of the weight of the concrete that acts normal to the axis of the arch. If accurate results are desired, it is necessary to make an exact analysis of the stresses in the arches, as

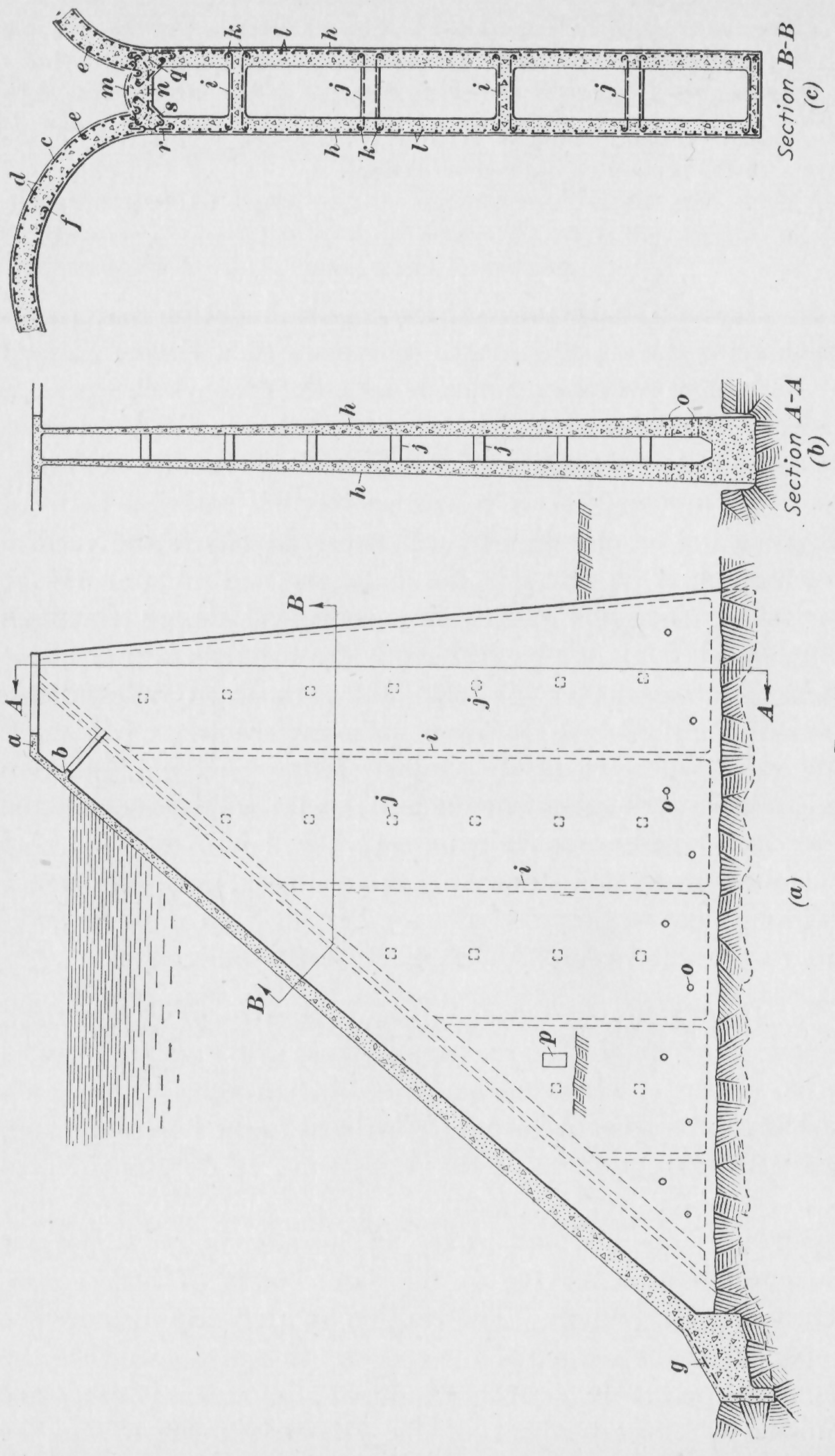

Fig. 31

described for a single-arch dam, and to consider the exact distribution of the loading. In the usual designs, it is assumed that the arch is subjected to a uniform load equal to the mean water pressure between the crown and the abutments. In establishing the tentative dimensions, the cylinder formula given in Art. 62 is applied, and the weight of the concrete is disregarded. The stresses due to shortening of the arch and variations in temperature of the concrete should be investigated carefully at all levels. Wherever tensile stresses are liable to occur, ample steel reinforcement should be provided.

The arches should be so shaped that the neutral axis of each section will coincide closely with the line of pressure corresponding to the loading. In general, three-centered curves are suitable for the sections in the upper part of the dam and simple circular curves may be used in the lower part. If desired, additional strength to take care of the effect of the non-uniform loading near the top of the dam may be provided by means of an auxiliary rib at the crest, as at *a* in Fig. 31 (*a*), and at a section about 10 feet below the crest, as at *b*.

In Fig. 31 (*c*), the main reinforcing bars *c* and the secondary bars *d* in the arches are arranged in two layers, one near each surface, but in some multiple-arch dams these bars are placed in a single layer that is near the up-stream surface at the buttresses and near the down-stream surface at the center of the span. Some of the main bars are continuous for the entire arch, and additional short bars, alternating with the long bars, are inserted midway between them near the up-stream surface at the buttresses and near the down-stream surface at the center of the span; the hooked ends of these short bars are shown at *e* and *f*. In order to take care of any cantilever action that may be developed at the base of the dam, the lower parts of the arches should be reinforced by means of the bars *g* in view (*a*), which are well anchored in the foundation bed.

80. Details of Buttresses.—In the early multiple-arch dams, the buttresses were made solid. As such buttresses were only a few feet thick even for very high dams, they were usually stiffened by means of horizontal struts similar to those

shown for the deck dam in Fig. 3. However, hollow buttresses, as shown in Fig. 31, later came into use. Each such buttress consists of two face walls *h* connected by cross-walls *i* and cross-struts *j*, located as in view (*a*). In this manner, a comparatively great total thickness, or distance between the outside faces, is obtained economically in each buttress, in which the cross-walls serve as webs so that the entire buttress may be treated as a series of **H**-shaped and channel-shaped columns. As shown in view (*b*), the thickness of each face wall increases toward the bottom as the compressive stress in the buttress increases. The total thickness of the highest buttress of the dam is generally $\frac{1}{15}$ to $\frac{1}{10}$ of its height, and the distance between cross-walls should not be more than about 12 times the thickness of the face walls at the bottom of the buttress. The buttresses are further stiffened by the two heavy vertical reinforcing bars, *k* in view (*c*), that are placed in each face wall at each cross-wall and cross-strut; the horizontal reinforcing bars in the cross-walls and cross-struts are hooked around the bars *k*. The face walls are also reinforced near the outside surfaces with two sets of bars *l*, one set running vertically and the other set running parallel to the axes of the deck arches. These bars stiffen the buttresses and also help prevent shrinkage cracks in the concrete. In the buttresses of some of the larger multiple-arch dams, additional protection against shrinkage is provided by means of contraction joints.

The end of the buttress that is exposed to the water may be either arched, as shown in view (*c*), or straight. When the end is arched, the size and spacing of bars *m* are the same as for the long bars *c* in the main arches; the bars *n* are provided to resist any unbalanced horizontal thrust that might occur in the small buttress arch if a main arch should fail. Small openings, *o* in views (*a*) and (*b*), through the buttresses are provided below the stream bed to prevent unbalanced pressure on a buttress from water that may seep under the dam. Also, an opening for a passageway, as *p* in view (*a*), is left in each buttress.

In designing the buttresses, it is customary first to assume trial dimensions, and then to investigate the stresses produced and to make any necessary changes in the thickness of the face walls and in the down-stream slope of the buttresses. The compressive stresses in the buttresses may be calculated by proceeding as explained for the buttresses of a deck dam, the bearing area of the deck arches on the foundation bed being neglected. Between the buttress and the arches is usually provided a construction joint, *q* in view (*c*). In order to bind the arches and the buttress together, tie-rods *r* and *s* are inserted.

EARTH DAMS

GENERAL FEATURES OF CONSTRUCTION

FOUNDATIONS FOR EARTH DAMS

81. Preparation of Foundation.—An essential requirement in the construction of an earth dam, such as that shown in Fig. 4, is the provision of a foundation of suitable material and formation so that seepage under the dam will be slight and there will be no danger of the structure being undermined. The most desirable foundation material for an earth dam is a sandy or gravelly clay. However, other materials can be used if suitable precautions are taken in preparing the foundation. The first step is to remove from the entire area to be covered by the dam all sod, brush, trees, roots, and other perishable vegetable matter and all soil to a depth equal to that to which the roots penetrate. If the exposed material is sandy or gravelly clay, it is only necessary to score the surface with a plow in order to provide a bond between the foundation and the dam. In case solid rock is encountered in the foundation, concrete cut-off walls running parallel to the axis of the dam and extending about 12 inches into the rock and rising 2 to 4 feet above the surface of the rock should be built at intervals of about 20 feet. In a foundation of open sandy or gravelly material, a cut-off trench for the core wall must be excavated down to an impervious underlying stratum.

82. Springs on Foundation Area.—Where running springs are found on the foundation area, they should be stopped up with concrete if possible. Strong springs in the down-stream portion of the embankment may be piped to discharge outside

of the area covered by the dam, but this procedure is not considered safe for springs in other locations. If the springs are likely to cause trouble the site may have to be abandoned.

METHODS OF DEPOSITING EARTH

83. Rolled-Fill Method. — The three methods generally used in depositing the earth in an earth dam are the rolled-fill method, the semi-hydraulic-fill method, and the hydraulic-fill method. In the rolled-fill method, which is employed mainly for small dams or for those of ordinary height, the material is hauled to the embankment in cars, trucks, dump wagons, or scrapers and spread in layers that are usually horizontal and from 6 to 12 inches deep. All the material should be uniformly moistened either by wetting in the pit from which it is excavated or by sprinkling on the embankment. The full thickness of each layer should be moistened only to such a degree that the maximum compactness will be obtained by rolling. An excess of water is to be avoided, as it increases the amount of settlement that takes place after rolling is completed. The roller should weigh between 200 and 300 pounds per linear inch of tread. If the rolled surface of any layer is too smooth for a proper bond with the next layer, it is roughened or loosened by harrowing.

84. Semi-Hydraulic-Fill Construction.—In the semi-hydraulic-fill method, dry earth is hauled to the embankment and dumped along the edges to form levees. Water is then admitted to the space between these levees to form a pool, called a *segregation pool,* and the dry material is washed toward the center of the pool by means of water jets mounted on barges. The coarsest particles that are loosened by the jets settle out first and are deposited near the outer faces of the final embankment, while the finest material is carried farthest and deposited near the center of the dam. Hence, the dam thus formed consists of a central portion, or core, of fine impervious material that is backed up on both sides by coarser and heavier material. The material in the core becomes consolidated as the water in it drains off toward the edges of the embankment.

During construction the edges of the embankment are kept higher than the central portion in order to retain the segregation pool. To keep the level of the pool from rising too high, pipes or flumes are usually provided for drawing off the excess water.

85. Hydraulic-Fill Process.—Where conditions are suitable, the hydraulic-fill method may be advantageously used. In this process, the material for the dam is excavated from its natural position by means of water jets, and the water-borne material is conveyed through a flume or a pipe to the edges of the embankment where the coarser material is deposited, and, as the water flows toward the center of the embankment, the finer particles of earth gradually settle out. The embankment is started by constructing low dikes of coarse material at the edges, and the mixture of water and earth is delivered inside of these dikes. Drainage of the surplus water is accomplished as in the semi-hydraulic-fill process.

When the source of the material for the dam is high enough above the dam to permit a grade of 6 per cent. for the sluicing flume, the mixture of earth and water may flow to the dam by gravity. Otherwise, the mixture has to be pumped. The water that drains off from the embankment can be returned to the pumps by a special arrangement of flumes or pipes.

The advantages generally realized by the hydraulic-fill process are speed and economy in construction, but this method does not produce satisfactory results where the material is a fine clayey earth which does not drain readily.

DETAILS OF CONSTRUCTION

86. Simple Embankments.—Small dams built by farmers to impound water for their stock and for domestic use are generally simple earth embankments without special cores. Such an embankment can also be used where sufficient impervious material is available to form the entire dam. The material commonly used is a mixture of loam, sand, gravel, and clay, the proportions depending on the source of supply. In order to obtain an impervious embankment, a certain amount of clay is necessary, but an excess of clay reduces the stability of the dam.

Where a simple earth embankment without a special core is desired and there is not enough impervious material to form the entire dam, the best material is selected and placed where it will be most effective in preventing seepage. The best position depends largely on the method of construction. If the character of the material and the size of the dam are such that the hydraulic-fill method can be adopted and the work is done under competent supervision, the best water-tight material is deposited by the water in a puddled core at the center of the embankment. Where the embankment is small and the rolled-fill method is considered preferable, satisfactory results can usually be obtained by placing the selected water-tight material in the up-stream portion of the dam and constructing the down-stream portion of heavy, stable, freely draining material, such as coarse sand, gravel, or stone. This material should be so distributed that the coarsest particles are at the down-stream face of the dam and the size of the particles gradually decreases until the impervious material is reached.

87. Types of Core Walls.—Earth dams are usually provided with special core walls. To be most effective, the core wall must extend downwards into an impervious stratum in the foundation bed and upwards at least to the high-water line. The core wall minimizes percolation through the embankment, prevents flow through pervious strata in the foundation, prevents damage from burrowing animals, and permits the use of less suitable materials for the main body of the embankment. Core walls are usually made of concrete or puddle, but may be constructed of rubble masonry, riveted steel plates, or wood. In a few cases loam or impervious top soil has been used.

Timber sheeting is a cheap material for core walls, but it is likely to decay rapidly and should not be used for permanent construction. Steel cores must be coated with asphalt for protection against corrosion and are not often used because of their high cost. Concrete has now superseded rubble masonry almost entirely. Concrete core walls are generally constructed of plain concrete, but reinforcement is sometimes provided in order to effect a saving in concrete.

The position of the core wall in the dam depends on the material used and on local conditions. If the wall is of masonry, it should be vertical and therefore is usually placed directly under the crest of the dam. Puddle walls are also commonly vertical and under the crest, but sometimes a wall of this kind is placed near the up-stream face and parallel to that face, with just enough filling outside of it to prevent the puddle from slipping when the water level is low.

88. Comparison of Concrete and Puddle Cores.—Burrowing animals can damage a puddle core but not a concrete wall. Also, although cracks may occur in masonry resting on a yielding foundation material, owing to unequal pressures in the dam, such cracks will usually become sealed with impervious earth material; whereas a hole or breach in an earth core will tend to enlarge rapidly. Furthermore, the masonry wall is well adapted to making connections with abutments, outlets, and foundations, and may readily be extended above the dam to form a parapet and thus provide extra freeboard. A puddle core, on the other hand, is flexible and therefore not affected by unbalanced pressures. The puddle wall does not require a firm, rock foundation, but may be built on any substantial material. Another advantage of the puddle wall is that there is a better union with the remainder of the embankment.

89. Construction of Earth Dam With Core Wall by Rolled-Fill Method.—When the rolled-fill method is used in constructing an earth dam with a puddle core wall, the core is compacted in thin layers; it is brought up to the ground surface before the main body of the embankment is started and is afterwards kept level with the embankment. When a concrete core wall is used, the wall is built slightly above the ground surface before any earth is deposited and is kept a few feet above the embankment as the construction of the dam progresses so that the concrete will set sufficiently to permit ramming the earth thoroughly against the wall. In case the material to be used in the embankment varies in quality, as is often the case, the finer and better particles should be placed next to the core wall on the up-stream side and the coarser particles should be

used on the down-stream side. The largest gravel or pieces of stone should be put along the faces.

90. Cores in Hydraulic-Fill Dams.—In the case of hydraulic-fill dams, or those constructed by means of the hydraulic-fill or the semi-hydraulic-fill method, the core is formed of the finest material that is carried by the water. As the fine material is depended on for water-tightness, it is important to have the core sufficiently thick at all parts. However, some of the coarser particles are often transported by the water farther than intended. Moreover, it is generally conceded that the cores of hydraulic-fill dams never dry out entirely and never have much stability, but will flow under moderate pressure. Hence, tongues of coarse material extend into the core and there is constant danger that such a tongue will reach entirely across the core. This will permit seepage and possibly may cause destruction of the dam. Therefore, frequent tests of the core material should be made during the construction of a hydraulic-fill dam.

Another trouble in hydraulic-fill construction is the possibility of a slide, which allows the water of the segregation pool to escape and wash away a large part of the embankment. This danger is greatest near the top of the dam because the portions of the embankment that retain the water are narrowest there. Also, there is more likelihood of a slide in a semi-hydraulic-fill dam than in a hydraulic-fill dam, because in the former type the material at the edge of the embankment is usually less porous than the adjacent portion of the dam nearer the core and therefore water tends to collect inside the less permeable outside layer. This water exerts a pressure against the layer which may result in a slide. Drainage of the central portion of the dam should be facilitated by the use of porous material in the outer parts of the embankment.

91. Protection of Slopes.—In order to protect the up-stream face of an earth dam from scour by waves and from ice action, a pavement is usually provided at least down to the low-water level. The paving material may be concrete poured in place, precast concrete slabs, cobbles laid closely by hand, or

riprap dumped into place. A pavement should not be laid until all settlement in the embankment has occurred.

The down-stream slope is usually sodded or seeded, but it may be covered with a layer of gravel or broken stone, especially in arid climates where grass cannot be grown readily.

92. Spillways for Earth Dams.—Nearly one-half of the recorded failures of earth dams have been due to overtopping caused by inadequate spillways. Not only must the spillway have ample capacity, but it must also be so located that any flow over it will not injure the dam. If possible, the spillway is located away from the dam, as in a notch or saddle discharging into the main stream below the dam. Sometimes the dam is of composite construction; that is, it has a spillway section constructed as in a masonry dam. In order to retain the down-stream faces of the earth embankments where they adjoin the spillway portion of the dam, heavy concrete wing walls should be built out from the ends of the masonry section. Similar walls may be provided to protect the up-stream slopes of the embankments, or else these embankments may be continued in back of the masonry as if there were no spillway. Where the cost of a separate spillway of the ordinary type is prohibitive or the valley at the dam site is so narrow that space for such a spillway is not available, a shaft spillway may be constructed. This type consists of a vertical shaft, whose intake is on the up-stream side of the dam and at the desired spillway elevation, and a tunnel that connects to the shaft and discharges into the stream below the dam.

DESIGN OF EARTH DAMS

93. Forces Acting on Earth Dams.—The forces acting on earth dams are the pressure and the weight of the water stored behind the dam, and the weight of the material of which the embankment is composed. However, since an earth dam is not a rigid structure, the stresses in it cannot be analyzed and no formulas can be derived for its design. Experience and the results of study and observation on other dams furnish the principal rules in establishing the dimensions.

94. Causes of Failure.—The usual cause of failure of an earth dam is overtopping; if the water flows over the dam it rapidly erodes or washes away the embankment. An earth dam may also fail if the water percolating through it attains sufficient velocity to carry away particles of the earth. In this case, a passageway is finally created, through which an increasing volume of water at a greater velocity can pass, resulting in eventual failure. If the water percolating or seeping through the dam collects near the down-stream face, there is always danger of sloughing or sliding of a portion of the dam on an inclined plane. Protection against excessive percolation by means of an impervious core, and the provision of good drainage at the toe of the dam are therefore important safeguards.

When the side slopes are such as to prevent slides along the faces, the weight of the dam is so great that failure due to sliding on the foundation is improbable. If there is a core wall, the dam may be considered to be composed of three parts: (1) the core wall, which gives water-tightness; (2) the up-stream portion, which keeps the core from moving up-stream when the water level is low and also protects the core from exposure to the impounded water; (3) the down-stream portion, which holds the core from moving down-stream. Since the water is expected to penetrate the up-stream portion and exert horizontal pressure on the core, the down-stream portion must provide all the required resistance to sliding, a factor of safety of 2 being sufficient. It may be assumed that the material in the down-stream portion is dry.

95. Top Width.—The top width of an earth dam, even if it is an unimportant one, should be not less than 10 feet; in the case of high and important dams, the top widths should be much more. A top width of 20 or 25 per cent. of the height to flow line is none too much. One rule sometimes employed calls for a width of 5 feet plus 20 per cent. of the height of the dam. When the top of the dam serves as a road, the width may be determined by the probable traffic.

96. Superelevation.—An earth dam should be high enough to prevent the tops of the waves from washing over the crest of the dam, even during flood flow and high wind. On most earth dams a superelevation or freeboard of at least 10 per cent. of the height of the dam is allowed, but the amount depends on the distance of wave sweep, and also on other factors, such as location, size of reservoir, and rapidity of run-off. On large important dams, the freeboard should never be less than 10 feet.

97. Slopes of Embankment.—The slopes of the up-stream and down-stream faces of an earth dam depend largely on the materials used and on the method of construction. On the up-stream face a slope that is just flat enough to prevent slides is usually satisfactory. Since the material is saturated with water, the slope of repose for wet earth must be considered. When the material is quite coarse, a slope of 2 horizontal to 1 vertical is generally satisfactory, but with finer material, the slope is commonly 3 to 1 and sometimes flatter. If the material on the down-stream face is porous, it will be practically dry. A slope of 2 to 1 is usually adopted.

The slopes may be continued uniformly for the full height of the dam, but on high embankments the lower portions are frequently made flatter than the upper portions. Also, banquettes or berms are often used to break up the slopes, as shown in Fig. 4. On the up-stream face, they serve as a better support for the paving and, on the down-stream face, they provide room for paved gutters which facilitate the removal of rain-water during heavy storms and reduce its scouring action. It is desirable to space the berms 30 to 40 feet apart vertically.

98. Design of Concrete Core Walls.—The dimensions of concrete core walls are based largely on experience and judgment and the recommendations of various authorities differ. Thus, some authorities specify that the top width of the wall should be not less than 4 feet, whereas others allow a width as low as 2 feet. A comparatively thin wall may be used when it is reinforced. Above the original ground surface, the sides of the core wall should be battered, as there is then less tendency for the earth and masonry to separate during settlement. A good slope for a plain-concrete wall is about 1 horizontal to

10 vertical. Below the original ground surface, the wall is placed in a trench, which should extend downwards into an impervious foundation, such as rock or hardpan. The sides of the trench are usually vertical.

99. Design of Puddle Walls.—The shapes and dimensions of the puddled cores in existing dams vary greatly according to the character of the materials used and the method employed in constructing the dam. When the core is built by the rolled-fill method, its thickness is usually made from 4 to 8 feet at the high-water level and about one-third of the depth of the water at the ground surface. In several large dams built by the hydraulic-fill method, the thickness of the puddle core at any level above the ground was made equal to the height of the dam above that level. The sides should be battered uniformly rather than stepped. The trench below the ground level should be wedge-shaped, as a tighter contact will then be obtained between the puddle and the sides of the trench than when those sides are vertical or stepped. At the bottom of the trench, a thickness of one-half of that at the ground level is recommended, provided it is not less than 4 feet.

DAM APPURTENANCES

CONTROL OF HEADWATER

100. Methods of Control.—When the maximum height to which the water impounded behind a dam may rise is not limited by land or water rights, it is satisfactory to take care of flood flows simply by allowing the excess water to escape through an unobstructed passage whose lower edge is at a certain fixed elevation. However, if such a spillway were used where the high-water level is limited, it would be necessary either to keep the crest of the spillway well below the high-water level and thus sacrifice storage capacity or head, or else to use a very long spillway. When the spillway cannot be made sufficiently long to permit a reasonable depth of overflow, it is necessary to use special devices for varying the discharge capacity of the spillway.

The various devices that are commonly used for controlling the head-water level may be divided into the following four general classes: crest controls, crest gates, sluices, and siphon spillways. Among the devices included under crest controls are temporary flashboards, permanent flashboards, and various types of gates.

101. Flashboards.—A simple and inexpensive method of raising the water level above the crest of the spillway is by means of flashboards, which are vertical boards or panels placed

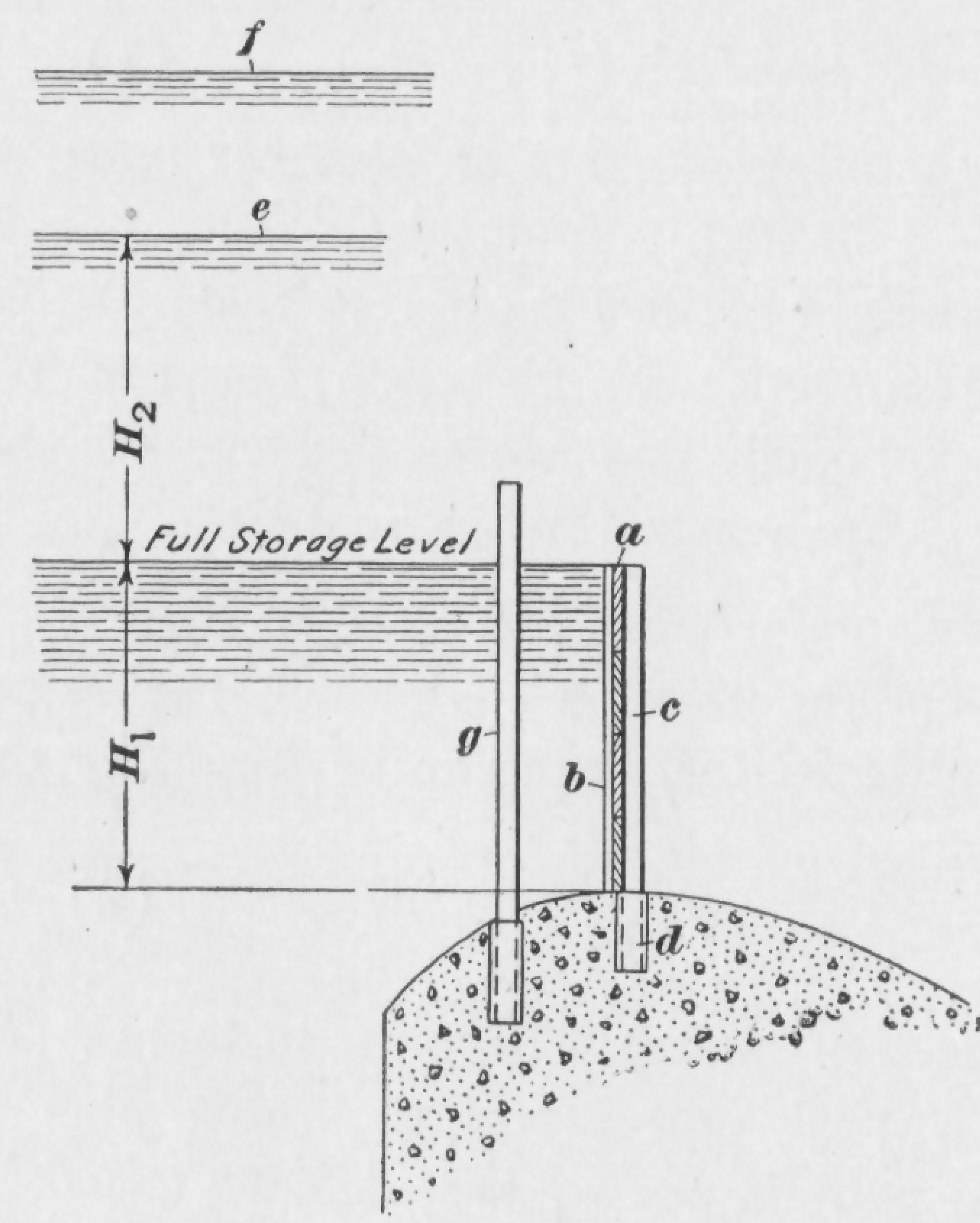

FIG. 32

on the crest of the spillway. There are two main types of flashboards, namely, temporary and permanent. Temporary flashboards are usually constructed and held in place as shown in Fig. 32. The boards are built up in panels by fastening planks *a* together by cleats *b*. The panels are held in place loosely by pins *c*, consisting of steel pipes or rods set in pipe sockets *d*. The size of the pins is such that when the water reaches a preestablished maximum height *e*, several feet above the top of the flashboards, the pins bend over quickly and the boards are washed away; the pins then lie flat on the spillway

crest and offer no obstruction to the overflowing water, which may rise to even a higher level f. The flashboards are replaced after the flood has passed. Sometimes the boards are chained to the abutments so that they can be recovered after having been washed out. In order to facilitate the restoration of the flashboards, it is advisable to provide pins g at intervals of 7 to 10 feet for mooring a barge.

In designing the pins for the support of the flashboards, the bending moment on them per linear foot of dam may be found by the formula

$$M = 125\, H_1^2 (H_1 + 3\, H_2) \qquad (1)$$

in which M = bending moment on pins per linear foot of dam, in inch-pounds;
H_1 = height of flashboards, in feet;
H_2 = difference in elevation between top of flashboards and water level at which flashboards are washed away, in feet.

The distances H_1 and H_2 are indicated in Fig. 32.

When the bending moment is known, the required section modulus per linear foot of dam can be found by the formula

$$S = \frac{M}{f} \qquad (2)$$

in which S = section modulus of pins, in inches^3;
f = flexural unit stress at which pins will fail, in pounds per square inch.

For steel pipe, the value of f has been found by test to be between 42,000 and 58,000 pounds. Values of the section modulus for standard steel pipe are given in Table I.

EXAMPLE.—Determine the required size and spacing of pins made of standard steel pipe for flashboards which are 4 feet high and which are to be washed out when the water rises 6 feet above their top; the value of f may be taken as 50,000 pounds per square inch, and the pins are to be spaced about 4 feet on centers.

SOLUTION.—By formula 1, the bending moment on the pins per linear foot of dam is

$$M = 125\, H_1^2 (H_1 + 3\, H_2) = 125 \times 4^2 \times (4 + 3 \times 6) = 44{,}000 \text{ in.-lb.}$$

Then, by formula 2, the required section modulus per lin. ft. is

$$S = \frac{M}{f} = \frac{44{,}000}{50{,}000} = .88 \text{ in.}^3$$

Since the pins are to be about 4 ft. apart, the section modulus of each pin should be approximately $4 \times .88 = 3.52$ in.^{3} If 4-in. pipe is used, for which the section modulus is 3.22 in.^{3}, the spacing may be $3.22 \div .88 = 3.66$ ft., or say 3 ft. 8 in. Ans.

TABLE I

SECTION MODULUS OF STANDARD STEEL PIPE

Nominal Diameter Inches	Section Modulus Inches^3	Nominal Diameter Inches	Section Modulus Inches^3
1	.133	3	1.73
1¼	.235	3½	2.39
1½	.325	4	3.22
2	.561	4½	4.16
2½	1.07	5	5.47
		6	8.50

Permanent flashboards are used only in special cases. They operate on the same principle as temporary flashboards, but the supports are designed so as not to be injured when the boards are carried away. It is then possible to replace the boards soon after the water has fallen to the level at which they are carried away and some of the flood water is thus conserved.

102. Gates for Crest Control.—Various types of gates for crest control have been employed. The type of gate used for the New York State Barge Canals, which is known as the *Stickney type,* is shown in Fig. 33. The movable part is hinged at a and consists of two legs b and c, the upper leg being somewhat longer than the lower one. While the water is below the limiting high level, the headwater pressure acting on the shorter leg c through the passage d holds the gate in its closed position, as shown in the illustration. When the water reaches a pre-established maximum height, the pressure on the longer leg b is sufficient to force the gate down until that leg becomes hori-

zontal at the level of the crest of the spillway. After the water level drops, the gate automatically rises to its closed position.

103. Crest Gates.—When flashboards or other crest controls are used, there are only two positions of the edge over which the water flows. Therefore, such devices are not suitable when it is necessary to confine the fluctuations of the water level within narrow limits. Where close control is required, a more intricate and expensive installation, such as a crest gate or a siphon spillway, must be used. Crest gates are supported by piers constructed at regular intervals across the spillway. They are raised and lowered, from a bridge spanning the spillway, by means of fixed or traveling hoists.

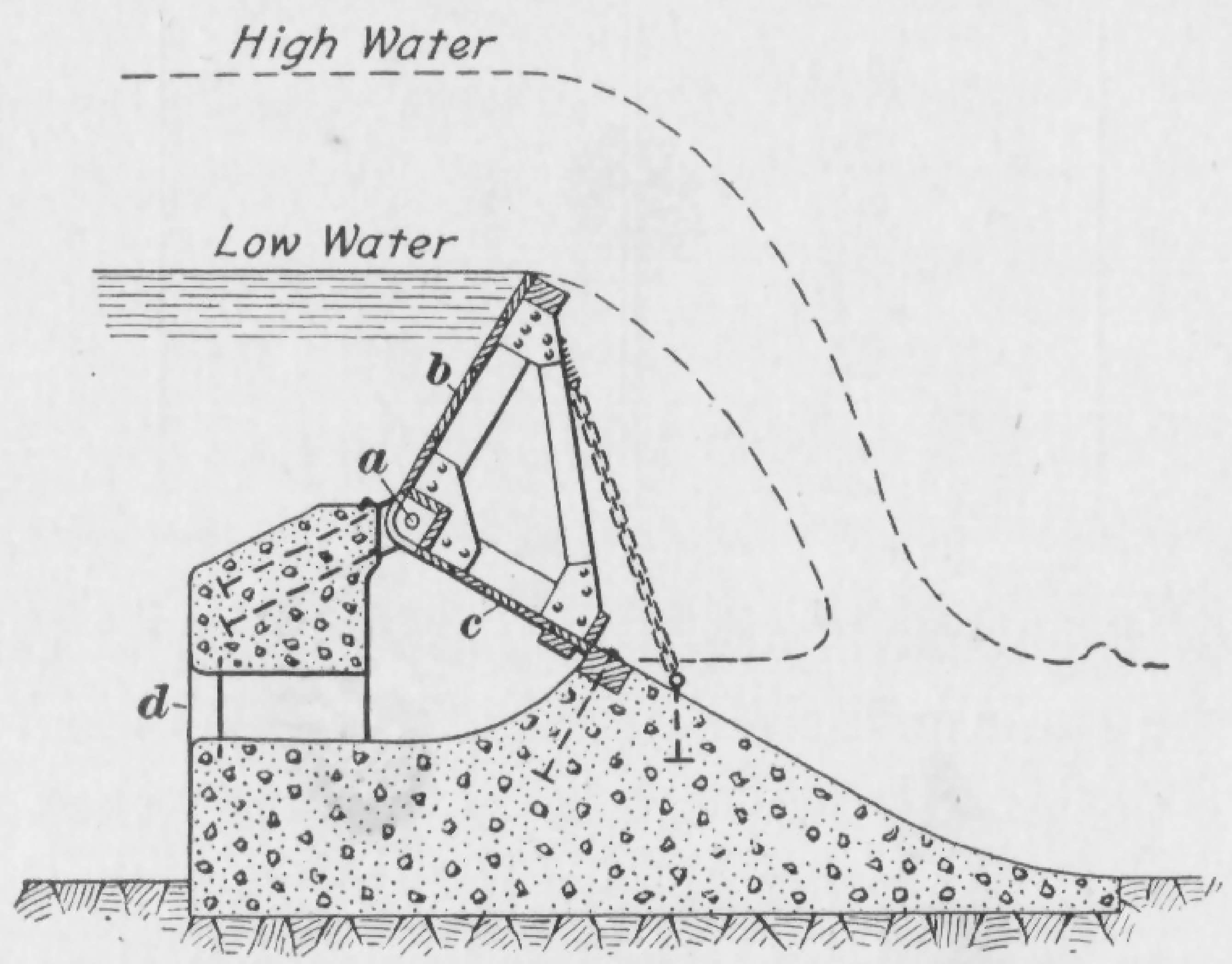

Fig. 33

In Fig. 34 is shown a type of crest gate, called the *Taintor gate,* which is widely used, especially where the water surface never rises above the top of the gate. The surface *a,* which is usually of steel but may be of wood, is curved to a true circular arc, and the pivot *b,* on which the gate swings, is located at the center of that circle. The gate can be raised to any desired position, as shown by dotted lines in the illustration, by means of the chain *c* which passes around the drum of a hoist at *d.* When the bottom of the gate is above the water level, the gate can close under its own weight.

The type of crest gate that is generally used for very large drainage areas and heavy flood flows, such as at the Wilson

dam on the Tennessee River and at the Gatun Spillway of the Panama Canal, is the Stoney gate. In this type, the gate, which is built up of structural steel, is guided in grooves in the

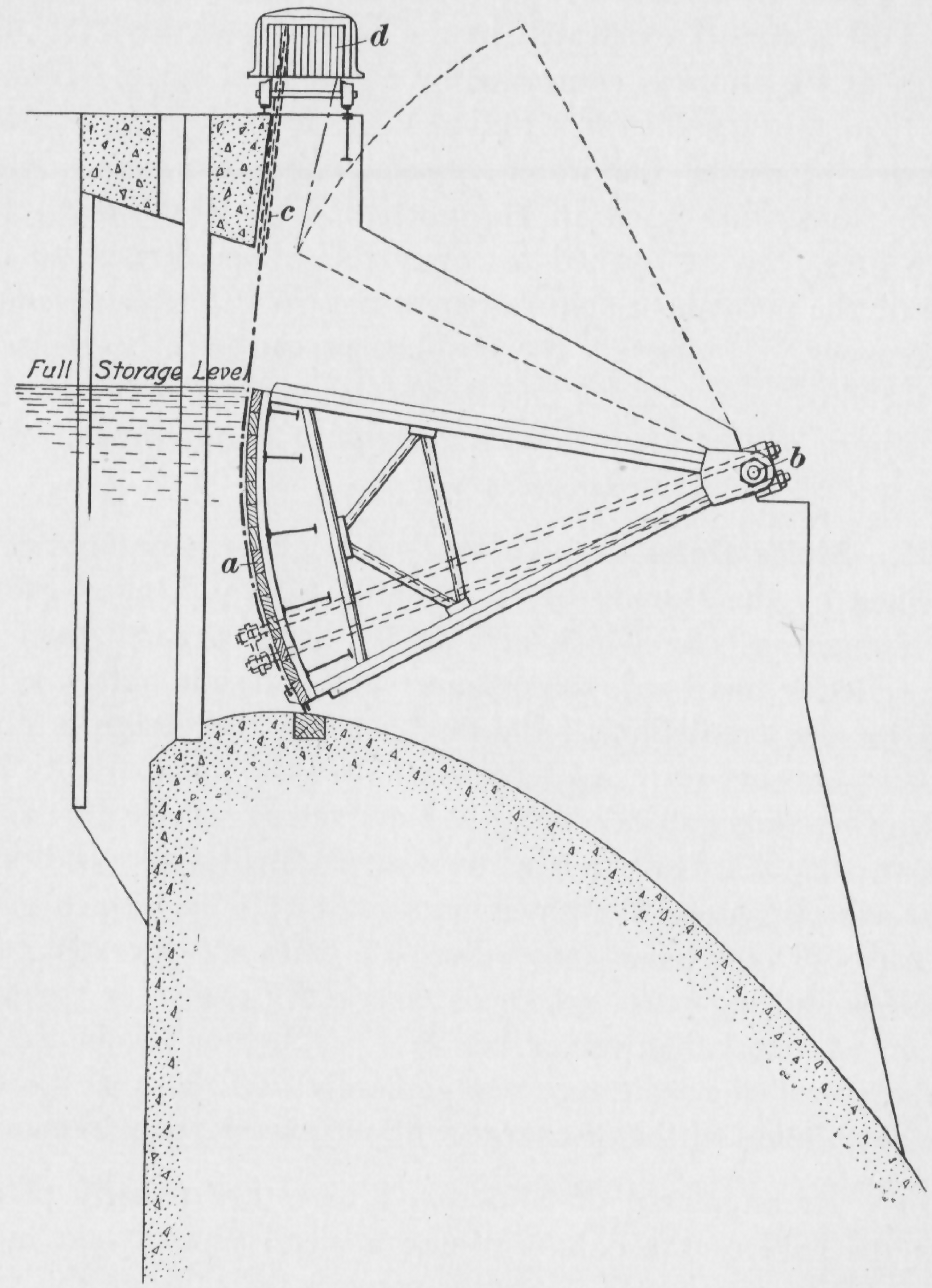

Fig. 34

masonry and bears on trains of hard-steel rollers traveling in the grooves. The rollers are not connected to the gate but move only half as fast as the gate. This type of gate also closes under its own weight when not subjected to water pressure.

104. Use of Sluices.—Where it is desired to draw off water from behind the dam without having the water flow over the crest of the dam, sluices are provided. The water is then conveyed either by means of a conduit through the dam or by means of a tunnel excavated in the natural soil, and the discharge of the sluice is controlled by a gate or a valve. Sluices are frequently installed in addition to devices for crest control, in order to utilize the water stored behind the dam. Also, where sluices are used in conjunction with flashboards, the sluice gates can be opened to lower the water surface to the level of the permanent spillway crest before all the flood water has escaped. If desired, the flashboards can then be replaced and the sluice gates closed, and the remaining flood water stored. Sometimes sluices are provided merely to empty small reservoirs in case repairs are necessary.

105. Sluice Gates and Valves.—A high-pressure outlet is classified by the Bureau of Reclamation of the United States Government as one which acts under heads greater than 75 feet. Below this head, the ordinary type of slide gate may be used for the regulation of the discharge, if the design is good and air vents are provided in back of the gate. Experience has shown that slide gates do not give satisfactory service for heads greater than 75 feet, as the unbalanced hydraulic conditions cause vibrations and the gates deteriorate rapidly. Such gates are used for high heads only where the gates are operated fully open for sluicing purposes, or as emergency gates for the protection of regulating valves below. For higher heads, valves of the so-called needle type are generally used, as they permit close regulation of the discharge without excessive maintenance.

106. Arrangement of Sluices.—Sluices are usually placed near the base of the dam. Where a metal pipe is laid in an earth dam or a concrete conduit is constructed through the dam, extreme care is necessary to prevent water from following along the exterior of the pipe or conduit. Cut-off collars and flanges should be placed at suitable intervals to prevent excessive seepage, especially in the core wall of the dam. In a concrete dam, a passageway can be readily constructed in the

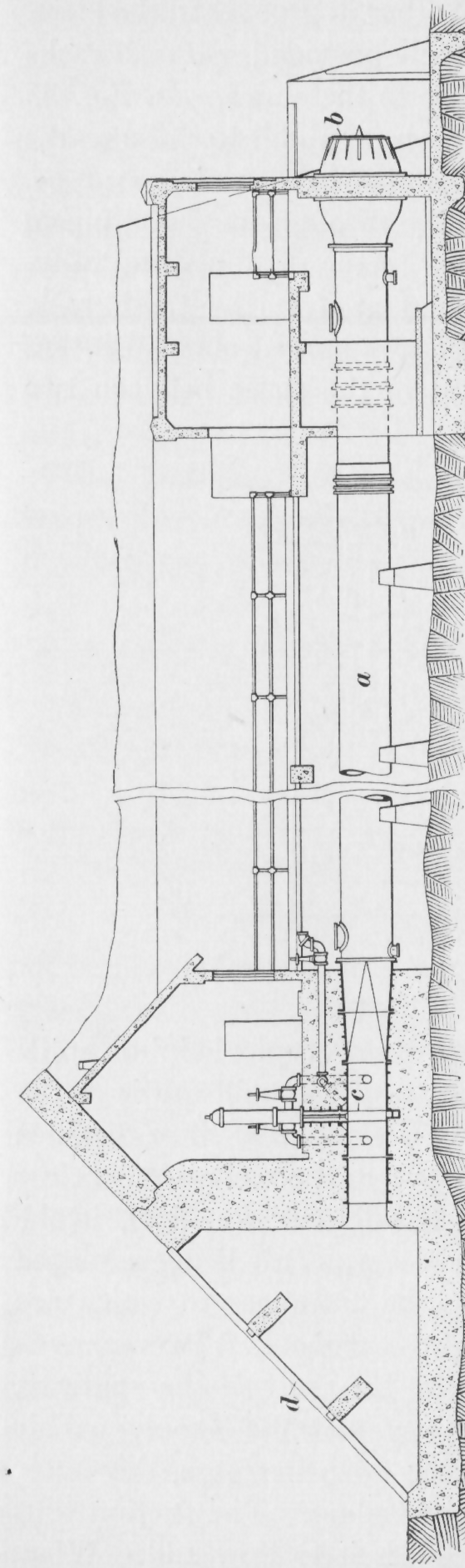

Fig. 35

masonry. However, metal pipes are often used, in which case special attention must be given to the contact between the metal and the concrete.

The control gate or valve is usually placed inside the dam. Owing to the fact that there is a sudden change in the section of the sluice at the control, which is conducive to rapid erosion, a masonry conduit should be lined with cast iron or steel just below the control. Also, the lining must be provided with ample drainage holes to prevent rupture due to the pressure of water that seeps from the reservoir. In the case of high-pressure sluices it is therefore advisable to place the control valve at the outlet, unless the spillway makes that location unsuitable.

There should be no sudden changes in the area of the sluice between the entrance and the control, and the section should be uniform from the control to the outlet. In order to permit repairs to the control, an emergency gate should be provided in the sluice above the control. On low dams, it is customary to install stop logs at the entrance to the sluice. These are loose timbers which span the opening between con-

crete piers and are laid one above the other in grooves in the piers. For high dams, a sliding gate is usually provided. Trash racks are generally installed at the entrance to the sluice. In Fig. 35 are shown sluice and control arrangements similar to those on the Stony Gorge Dam in California, which is of the Ambursen type. Under normal conditions, the discharge through the outlet pipe *a* is controlled by the needle valve *b*. At *c* is a hydraulically-operated emergency sliding gate, and at *d* is the trash rack, which is constructed of $\frac{7}{8}'' \times 6''$ steel bars spaced about 6 inches apart. The outlet works are placed in the space between two abutments.

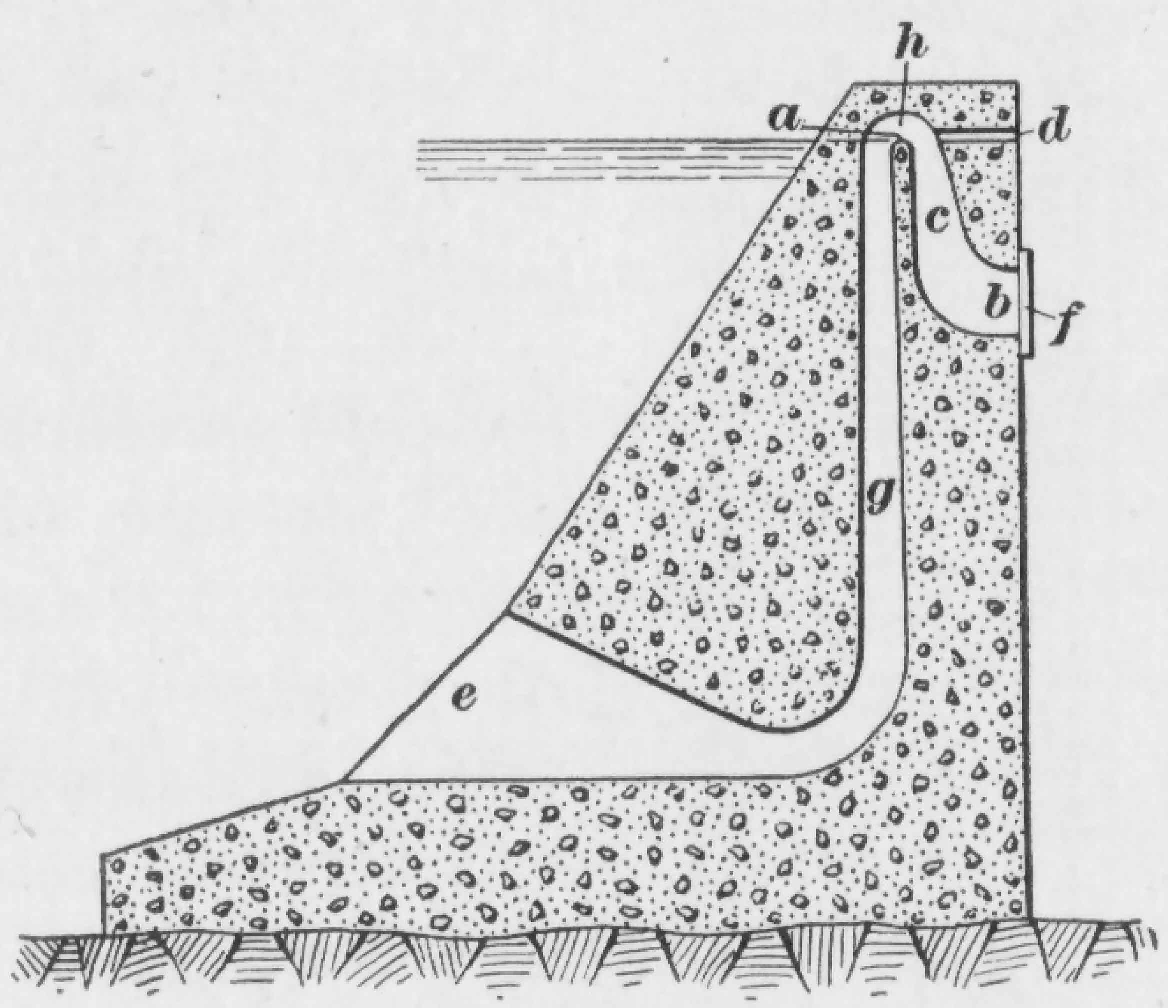

FIG. 36

107. Operation of Siphon Spillways.—A typical siphon spillway is shown in Fig. 36. The crest *a* of the spillway is at the high-water level in the reservoir and the inlet *b* is some distance below that level. When the water level in the reservoir is below the crest of the spillway, the water rises to the same level in the upper leg *c* of the siphon as in the reservoir, air being admitted to the siphon through the vents *d*, the entrances to which are at the level of the crest *a* or slightly above it. However, as the water in the reservoir rises above the crest of the spillway, flow commences through the spillway and the supply of air through the vents is cut off. As the flow increases, the water soon carries with it the air from the siphon. The suction thus produced primes the siphon and causes it to flow full. When

the continuous flow is established, the total head that causes flow is the vertical distance from the water level in the reservoir to the center of the outlet at *e*. Hence, the discharge through the spillway is considerably greater than that for an overflow spillway for which the head would be the height of the water above the crest *a*. When the discharge exceeds the flood flow, the water level drops until air is again admitted to the siphon through the vents *d* and the siphon action is broken. Therefore, the flow through the spillway continues only as long as the water surface is above the level of the crest *a*.

108. Design of Siphon Spillway.—In calculating the discharge, a siphon spillway may be treated as a very short pipe, in which the velocity at the smallest cross-section is assumed to be $.65\sqrt{2gh}$. Thus, the required area at the crest of a siphon spillway may be found by the formula

$$a = \frac{Q}{5.1\sqrt{h}}$$

in which a = area of smallest part of spillway, in square feet;
Q = discharge, in cubic feet per second;
h = total head on center of outlet, in feet.

The area of the inlet is made two or three times as large as that at the crest. Although the inlet is placed well below the high-water level to prevent the entrance of ice and drift, it is usually protected by a trash rack *f*, Fig. 36. The water may be discharged from the outlet either into the air or under the surface of a pool of water on the down-stream side of the dam. If the outlet is submerged, the head on the siphon is measured to the surface of the pool of water instead of to the center of the outlet. In order to obtain the maximum discharge, the lower leg *g* of the siphon should be as long as practicable. However, the capacity of the spillway would not be increased by making the head greater than 34 feet, because the water column would then be broken by air entering the siphon. When the velocity of flow is high, the siphon should be lined with metal, especially at the throat *h*, in order to prevent erosion.

OTHER APPURTENANCES

109. Fish Ladders.—Certain kinds of fish are in the habit of going periodically to the headwaters of the streams in order to spawn or feed. For that reason, it is required by some State laws that a fish ladder, or a means for permitting the fish to

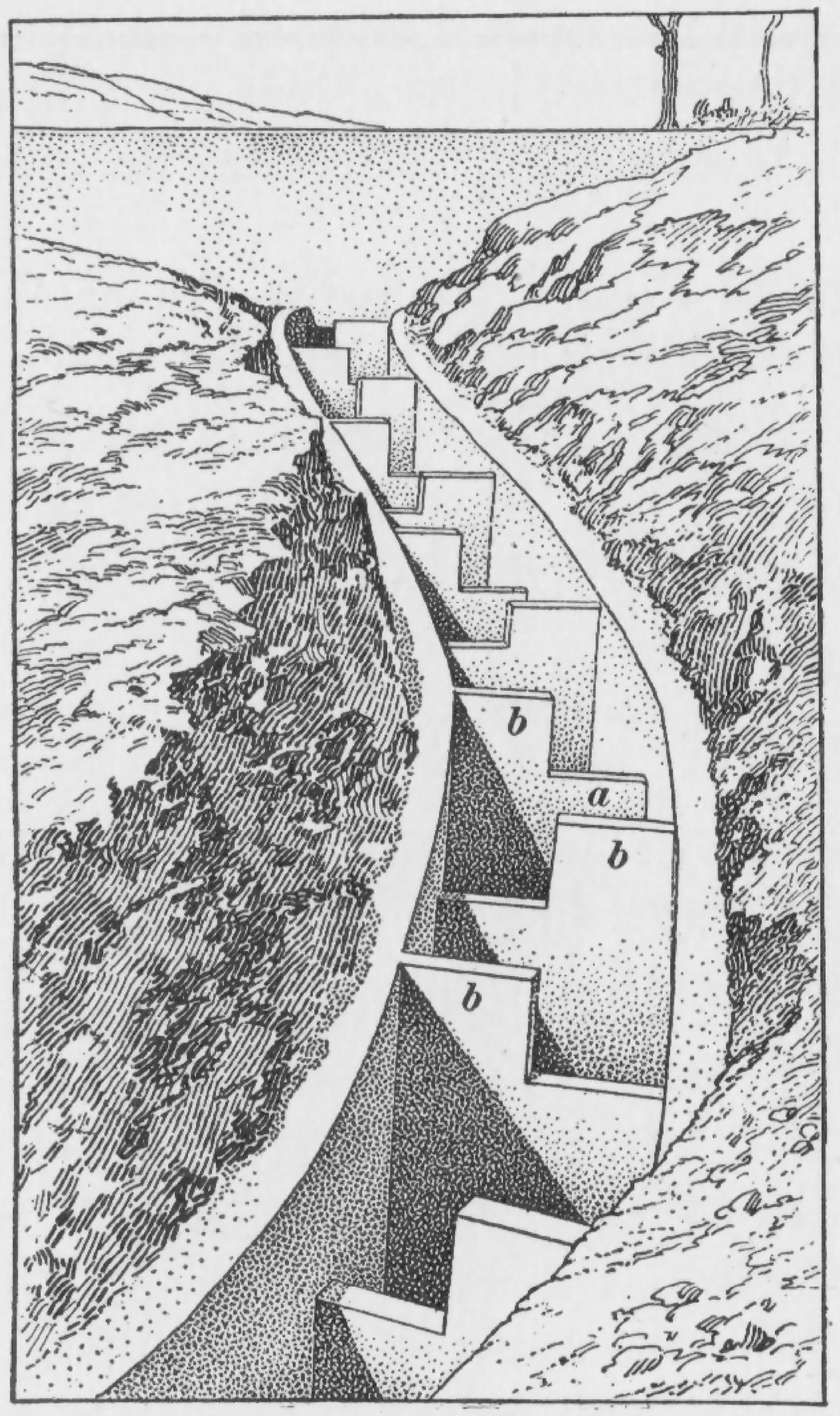

FIG. 37

pass safely up-stream, be provided wherever a dam is constructed across a stream. A fish ladder is an inclined trough or passageway in which the velocity of the water flowing from the upper to the lower pool is so reduced by a series of baffle walls that fish can easily swim against the current. There are three main types of fish ladders. In the so-called *overflow type,* shown in Fig. 37, each baffle b has either a **V**-shaped or rectangular notch a over which the water spills with a drop low

enough to permit the fish to jump over the notch. This type of ladder is not suitable for the more sluggish types of fish that will not jump over the notches. In the *sluice type* of fish ladder, each baffle is provided with a rectangular opening located near the bottom of the trough, and the fish can readily pass through these openings. For the third type, known as the *rapids type,* each baffle is made shorter than the width of the trough so that an opening is left at one end of the baffle for the passage of fish. In all three types, the openings in the baffles are staggered, as shown in Fig. 37, so that the water follows a circuitous course and its velocity is thereby reduced. Such staggering of openings also induces the fish to rest in each compartment between baffles before they locate the next opening.

110. Log Chutes.—When a dam is built across a stream in which logging operations are carried on, log chutes are required by law. The chute usually consists of an inclined flume whose inlet is an opening in the dam located below the low-water level. In order to conserve water, the flume is made as narrow as the size of logs permits and the amount of water entering the flume is controlled by a suitable gate.

IRRIGATION

Serial 2901 Edition 1

WATER SUPPLY FOR IRRIGATION

QUANTITY AND QUALITY OF WATER

FUNCTIONS OF WATER

1. Introduction.—Irrigation is the artificial application of water to the soil for the purpose of crop production. In most cases, irrigation supplements the natural rainfall of a region, being used when the rainfall is insufficient or occurs only at those periods of the year when crops are not benefited. In the United States, irrigation is employed mainly in the arid sections of the western states where, without it, large areas now producing valuable crops would be unproductive and valueless. In most arid regions irrigation is required from the middle of April to the middle of August, a period of about 120 days. Irrigation is also practiced, but on a much smaller scale, in the eastern part of the country in gardening and truck farming, and in nurseries, where the intensive cultivation methods justify the cost of irrigation works because of the greatly increased yield obtained.

2. Necessity for Irrigation.—Plants require a certain amount of moisture for their growth and development, this moisture being obtained by absorption from the soil through the roots. They also take up other substances from the soil. However, since plants cannot assimilate their food in a solid state, these substances must be in solution. The soil, therefore, must contain enough water to dissolve them. The maintenance of the

necessary amount of water in the soil around the roots of the plants is the primary purpose of irrigation.

3. **Conditions Suitable for Irrigation.**—Even when the average annual rainfall in a district is as much as 30 inches, which is more than sufficient for plant growth if falling opportunely, there may still be serious damage done by drought, possibly to the extent of losing certain crops, because rain may be lacking at a critical time during the plant growth. Two conditions are necessary for satisfactory crop production: first, the annual amount of water available must be sufficient; and, second, it must be distributed at the proper seasons. Where the normal rainfall does not meet these conditions, irrigation may be practiced. The extent to which it will prove desirable depends on the value of the crops, the damages caused by lack of rainfall, and the cost of an irrigation system.

4. **Advantages of Irrigation.**—Where irrigation is practiced skillfully, a much higher yield in crops may be expected than in areas where natural precipitation is relied on. It is possible to apply the water at the proper time and in the proper quantity for the best results, which is a matter of great importance with many crops. Moreover, irrigation acts as an insurance on the production of crops, eliminating the danger of crop failure due to too little rain; also, since an excessive amount of rain rarely if ever falls in the arid regions where irrigation is usually employed, there is little danger of too much water. In general, through the employment of irrigation, farming is put on a more exact basis and the returns are greatly increased.

5. **Economy of Irrigation.**—Although it is difficult, if not impossible, to give any general idea of the returns that may be expected from the construction of an irrigation system, a careful investigation will usually yield quite accurate data regarding the economic justification of a proposed project. The area of the land to be irrigated, the character of the soil, the kinds of crops that can be raised, the cost of the system, and the amount of water available during the irrigation season are most important factors in determining the advisability of the work.

The cost of a project includes not only the cost of the water rights, that is, the cost of providing the water for irrigation, but also the cost of leveling and preparing the land for irrigation. During recent years the cost of a water right on Government projects has varied from about $20 to $200 per acre. These costs are probably somewhat above the average, because Government irrigation projects are usually more elaborate and more difficult than most private projects. The cost of preliminary work on the land varies with the topography, the crops to be raised, and the kind of soil.

6. **Limitations of Irrigation.**—Irrigation alone must not be depended on to render arid soils productive. It is only one factor in the reclamation of otherwise uncultivable soil. With it must be combined the proper selection of crops, adequate underdrainage, and suitable preparation and cultivation of the land. Also, all other resources of ordinary farming must be employed.

7. **United States Reclamation Service.**—Increasing difficulty in carrying out large irrigation projects in arid regions of the United States by means of private capital led to the passage of the Reclamation Act in 1902. This act provided for the establishment of a revolving fund, called the Reclamation Fund, to be derived from the sale of public lands in sixteen Western states and used for the special purpose of investigating, constructing, and operating in those states large irrigation projects that could not be undertaken with private capital. The act made further provision that the reclaimed land be sold to settlers at a uniform price per acre that would be sufficient to reimburse for the entire cost of those projects, the payments to become available for further reclamation.

In accordance with the Reclamation Act, and in order to execute its provisions, the United States Reclamation Service was established and since its organization has carried out economically and efficiently vast irrigation projects under difficult conditions that could not attract the investment of private capital. Millions of acres of land in uninhabitable arid regions have thus been reclaimed and covered with productive fields and prosperous homes, villages, and towns.

DUTY OF WATER

8. Units of Measurement of Water.—For irrigation purposes, the volume of water may be expressed in various units. The cubic foot per second, often called second-foot for brevity, is perhaps the most generally used standard of measurement for flowing water. Another unit quite commonly employed in irrigation is the miner's inch. This unit, which had its origin in the old placer mining practice of the West, represents the discharge from an orifice 1 inch square under a certain head. In different states, differing values of the miner's inch are used, but ordinarily it varies from $\frac{1}{40}$ to $\frac{1}{50}$ second-foot. Because of this variation in value, it is preferable to use second-feet for water measurements involving the volume of flowing streams. For storage and other purposes where the time element does not enter, the acre-foot is most generally used. This is the amount of water that will cover 1 acre to a depth of 1 foot. Since an acre contains 43,560 square feet, 1 acre-foot is equivalent to 43,560 cubic feet.

9. Meaning of Duty of Water.—The term *duty of water* means the area that can be irrigated by a unit quantity of water. It is most commonly stated as the value obtained by dividing the area irrigated by the quantity of water used during the irrigating period. If the water is measured in acre-feet, and the area is measured in acres, the duty is expressed in acres per acre-foot. Thus, if 4 acre-feet can irrigate 2.2 acres, the duty is $2.2 \div 4$, or .55 acre per acre-foot.

10. Required Quantity of Water.—The amount of water required for irrigation is affected by various factors, among which are the following: the length of the irrigation or growing season; the amount of rainfall on the section, and its distribution; the kind of crops raised; the manner of preparation of the ground; the kind of soil; the care with which the water is used; and the losses in transportation and delivery. The most important factor is the skill of the irrigator, and next in importance is the kind of soil. For some crops, a 12-inch depth of water, or 1 acre-foot per acre, may be sufficient, and if the transportation losses are small, comparatively little excess water will be required. Thus, the duty is but slightly less than 1 acre per acre-foot. In other places, 24 inches of water may be required to produce crops, and where the losses in reservoirs and canals are large, considerably more water is necessary. In this case, the duty may be only about $\frac{1}{3}$ acre per acre-foot.

Extensive records of the amount of water used on a number of projects have been kept by the United States Reclamation Service. These records indicate an average duty, based on measured delivery to the individual farms, of somewhat more than $\frac{1}{3}$ acre per acre-foot; but a number of these projects have been operated consistently with a duty of nearly .7 acre per acre-foot. By a careful study of local conditions, in which the factors mentioned in the preceding article are considered, a reasonably accurate estimate of the probable duty of water can be made for any particular project. The total quantity of water, in acre-feet, required to irrigate a certain district is found by dividing the area of the district, in acres, by the duty, in acres per acre-foot, for the proposed crop. For example, if it is required to irrigate a district whose area is 10 square miles, or 6,400 acres, and the duty of the water actually applied to the irrigated land is expected to be $\frac{1}{2}$ acre per acre-foot, the total quantity of water to be delivered per year to the land is $6{,}400 \div \frac{1}{2} = 12{,}800$ acre-feet.

QUALITY OF WATER

11. Impurities in Water.—The standards for quality of water for irrigation are neither so high nor so rigid as those for water for domestic use. There is little or no objection to color, turbidity, and organic content in water for irrigation. On the contrary, some waters carrying large amounts of suspended matter have a certain value in irrigation because of the fertilizing value of the substances held in suspension. However, this material, though beneficial to the land, sometimes proves troublesome by obstructing channels and waterways and filling up reservoirs with silt or sediment. Strongly acid water, or water from mines, is not desirable for irrigation. Also, waters that carry dissolved salts in amounts exceeding 3,000 parts per million are objectionable; if carbonates are present, a still lower limit is imposed,

and even then, special measures must be taken to prevent a decrease of fertility.

12. Temperature of Water.—An important factor in the value of water is its temperature. The warmth imparted by a water of relatively high temperature is often sufficient to stimulate plant growth. The quality of the water in a stream or lake is sometimes indicated by the plant life in the water or along the banks.

SOURCES OF WATER SUPPLY

INVESTIGATION OF SUPPLY

13. Main Problem.—The two sources of water supply that are commonly investigated in studying an irrigation project are surface water and ground water. With either source of supply, it is often necessary to provide a reservoir for storing water in order to prevent waste when the water is supplied faster than it can be used and to hold the excess for the time when the demand for water is greater than the normal supply. In the investigation of a source of supply, the main problem is to decide whether the annual yield is sufficient to provide the quantity of water required for irrigation. In estimating the available yield, liberal allowances should be made for the various losses that will occur.

14. Important Considerations in Selecting Source of Water Supply.—In general, the problem of providing water for irrigation is similar to that of developing a water supply for a city, but there are some points of difference that should be noted. In the first place, as has already been mentioned, the question of sanitary quality is rarely of importance in irrigation. The chemical characteristics of the water have, it is true, some bearing on its fitness for irrigating purposes, but, broadly speaking, neither biological nor chemical examination plays any prominent part in the selection of the source of supply. In the second place, since irrigation is practiced mostly in districts where the rainfall is abnormally small, general rules regarding the supply derivable per square mile of drainage area are less applicable than they are for districts of average rainfall and evaporation.

In investigating an irrigation project for a section, more attention must be paid, and more weight given, to measurements and observations of the available supply, at least until a good general knowledge has been acquired concerning conditions in that section. In general, it will be necessary to make more careful measurements because of the relatively small amounts involved. On the other hand, in making estimates for irrigation projects, the engineer is not bound so rigidly as in the provision of a water supply for municipalities. In such a case, any failure in the daily supply of water may produce dangerous results, or at least considerable inconvenience, whereas failure to supply the full quantity required for irrigation will be attended by much less serious consequences.

SURFACE WATER

15. Run-Off.—The amount of water falling on a watershed in a year is equal to the product of the area of the surface and the depth of the annual precipitation. However, a large percentage of the rainfall is lost by absorption, evaporation, and other causes, and only the run-off is available for irrigation. The run-off in a given time is the quantity of water that flows off the surface of the land in that time and finds its way into the streams; it includes not only the water flowing from the surface immediately after rainfall, but also that derived from springs and similar sources. The best method of determining the run-off is by actual measurements of the stream flow. These measurements are especially valuable when they extend over a period of years, as the annual total supply of the stream varies considerably from year to year.

16. Records of Stream Flow.—In many countries and in some states of the United States, systematic gaging of streams is carried on. Reports of run-off on various watersheds in the arid regions have also been published by the United States Geological Survey. Valuable data may sometimes be obtained for estimating the run-off of a stream for a watershed under consideration either from records for that stream or from those for a neighboring stream having a similar watershed. How-

ever, as wide variations exist, not only between different streams, but also between different reaches of the same stream, it is necessary to make a careful study of the stream or streams in question in order to obtain accurate data concerning the flow at each point where the water is to be diverted for irrigation or where a reservoir is to be constructed. It is rather unusual to find systematic records of stream flow that have been kept in advance of an irrigation project. Therefore, it is often necessary to carry out a considerable amount of stream gaging in order to obtain the required data for the study of the project.

17. Gaging Stream Flow.—The two principal methods of gaging the flow of a stream are by a weir and by a current meter. The method to be adopted is usually determined by the character of the stream, its cross-section, and the material composing its bed and banks.

If the stream is not too large, and the bottom and banks are fairly firm, a dam may be constructed, in which an overflow weir is placed. When great accuracy in the calculation of discharge is not required, or where the observations are affected by factors—such as leakage around the dam, inaccuracies of measurement, or uncertainty in regard to the velocity of approach—that render the result doubtful, the following simple formula, in which the mean value of the coefficient of discharge is assumed, may be used.

$$Q=\frac{10}{3}bH^{\frac{3}{2}}$$

in which Q = discharge, in second-feet;
b = length of weir, in feet;
H = head on crest of weir, in feet.

Accurate results from the use of current meters can be obtained only where the channel is fairly straight, smooth, and regular, with little change in slope or form for at least 100 feet above and below the gaging station. The meter may be suspended from a bridge without piers, from a small platform or car running along a wire stretched across the stream as shown in Fig. 1, or from a boat anchored to a cable. The discharge

of the stream is computed from the meter readings and the measurements of the cross-section of the stream at the gaging station.

18. Discharge Records.—In order to have a complete record of the flow of the stream, it is necessary to make discharge measurements at various stages of height. The stage of the stream may be recorded by a gage board. When such complete data have been secured, a discharge curve may be prepared from which the run-off can be obtained directly from the gage reading. The accumulation of enough data for preparing such a curve requires a considerable period of time, and where the channel is shifting, frequent checking will be necessary.

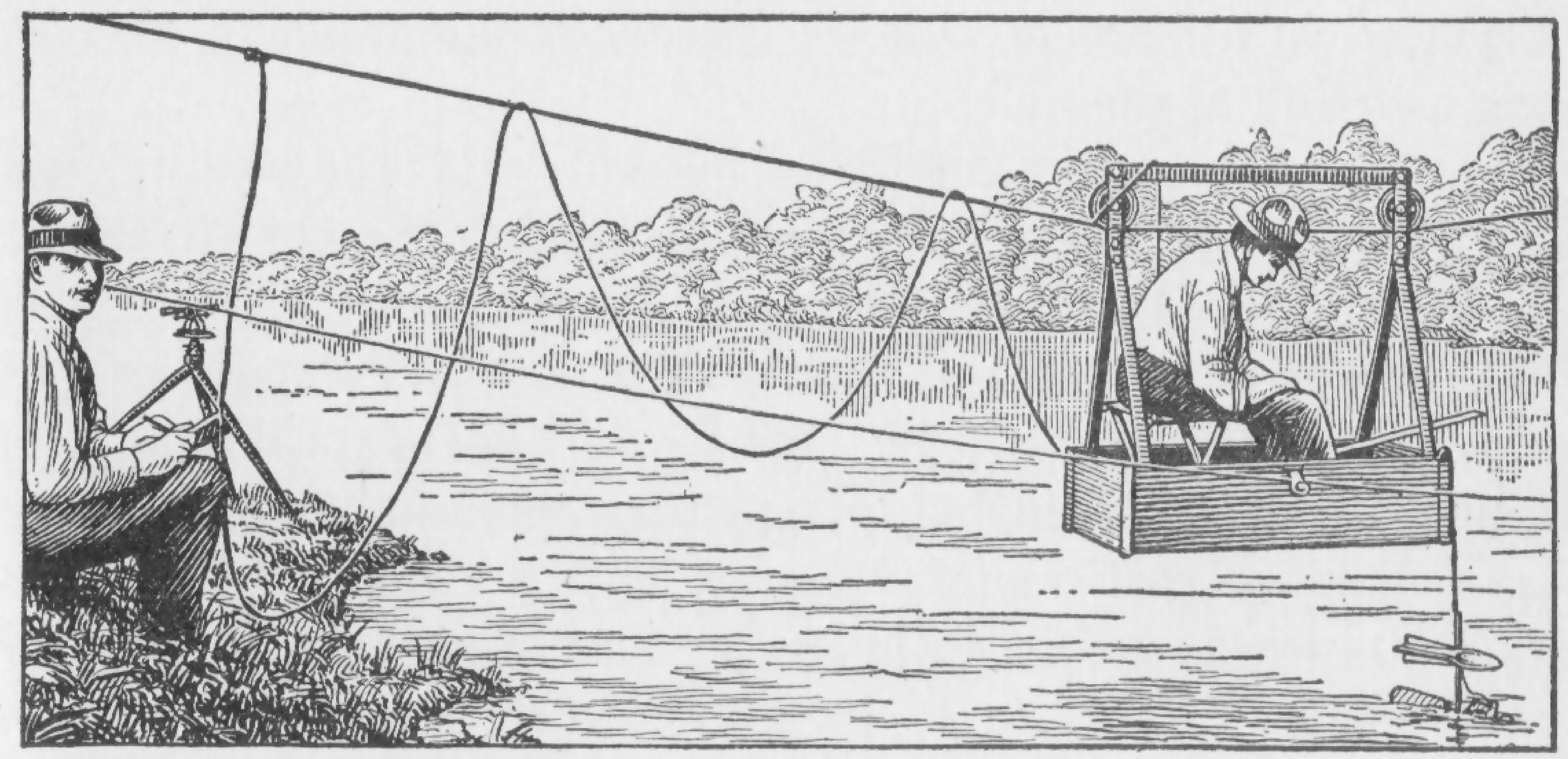

Fig. 1

Daily observations should be recorded for at least a year in order to obtain even an approximate knowledge of the normal flow of the stream. If the flow fluctuates markedly, readings should be taken twice a day. The time required to obtain an intelligent idea of the run-off from a watershed may constitute an embarrassing delay in those localities where such observations have not previously been carried on.

19. Study of Catchment Area.—In case previous records of stream flow are not available, and it is not convenient to make stream gagings, the probable run-off is sometimes estimated from the area of the watershed, the rainfall, and the losses. If

topographic maps of the territory including the watershed are available, the boundary of the watershed that is tributary to the reservoir site or the stream above the point of diversion can be located from the contours by following along the dividing ridges between the drainage or catchment basin under consideration and the adjacent basins. The area of this watershed can then be found from the map. When suitable maps cannot be obtained, the line bounding the catchment area may be located by running a survey along the crest of the watershed. As this survey will necessarily be approximate, no time or money should be wasted in refined instrumental work.

20. **Rainfall.**—Previous records of rainfall should be consulted, if any have been kept. Otherwise, a rain gage should be placed in a suitable location and observations made over as long a period as possible.

The amount of rainfall varies not only with the locality, but also with the seasons, and is often materially different in regions not far apart. In the western part of the United States it is much less in the flat plains than on the high and wooded mountains that may separate the plains. In and near Sacramento, California, it is 15 inches per year, whereas a comparatively short distance east, along the crest of the Sierras, it averages 50 to 60 inches. Still a little further east in Nevada the precipitation ranges from 5 to 10 inches per year. Near Yuma, Arizona, the rainfall during the growing season averages only about 1 inch. Generally, the rainfall increases with the altitude in the Sierra Mountain section at the rate of about 6 inches per 1,000 feet of elevation. In the upper Missouri and Yellowstone valleys in Montana, the average annual precipitation is between 12 and 20 inches, but of this amount only about 5 inches falls during the growing season. In the Platte and Arkansas valleys of Colorado, about 8 inches of rain falls in the growing season out of a total of about 15 inches.

21. **Estimating Run-Off.**—The run-off may be taken as a certain percentage of the total rainfall on the watershed; that is, a percentage of the product of the area of the watershed and the depth of precipitation. However, the chief difficulty is in

choosing the proper percentage, as it depends on many factors, such as the soil and topography of the watershed, the amount of vegetation on it, and the character of the rainfall. Thus, rock or clay, either bare or overlaid with a thin layer of soil, absorbs less of the rainfall than a deep bed of sand. Also, the steeper slopes shed more of the rainfall and, in general, vegetation tends to retain some of the precipitation and decrease the run-off. These and other factors materially affect the relation between the run-off and rainfall, and they should be carefully weighed in establishing that relation. A study of a neighboring catchment basin of similar area, topography, and altitude, for which records of rainfall and stream flow are available, is a great help in estimating the probable percentage of the total rainfall that may be assumed as run-off.

GROUND WATER

22. **Source of Ground Water.**—A part of the rain falling on the surface of the ground runs off as surface water, but most of it is absorbed by the soil, and passes through it downwards to lower strata, where it collects. This accumulation of water forms a great underground reservoir which is called ground water; its surface is the ground-water level or watertable. Under the influence of rainfall or drought, this water level rises or falls, usually within comparatively narrow limits.

Ground water may reappear naturally as springs or seeps, or it may be reached by collection works, including wells, tunnels, infiltration galleries, and subsurface collection systems. The amount of water which may be collected by such works depends on the amount of rainfall, the character of the subsurface strata, the velocity of flow of the ground water, and other factors.

23. **Quality of Ground Waters.**—As the water that is taken up by the ground flows through the strata of soil, it dissolves substances from the subterranean structures through which it passes. Therefore, while suspended matter has been strained out and ground waters are ordinarily clear, they may have taken up, during the underground travels, substances which render the water unsuitable for irrigation. Although this possibility does

not usually materialize, such waters generally should be examined to determine their chemical content.

Ground waters are usually quite cold, which is an undesirable factor in irrigation; but a short period of storage under normal conditions results in a rise in temperature.

24. Artesian Wells.—When water flows from a drilled well by gravity, the well is called an Artesian well. In such wells the water rises considerably above the stratum in which it was confined before the well was drilled, and does not have to be pumped. Artesian wells of large flow are not common in localities requiring irrigation. However, if an adequate source of supply should be secured from Artesian wells, it is only necessary to transport the water to the area to be irrigated.

25. Pumping From Wells.—When ground water is used for irrigation, it is usually necessary to resort to pumping in order to raise the water to the required height. If the water is to be delivered to an elevated reservoir, or is to be pumped some distance, a force main is needed. When the water from a single well is to be pumped, the problem is not a difficult one; but, when there is a group of wells, the pumping system is more complicated. If electric power is available, a small motor-driven pump may be installed at each well; where other power must be used, a central station may be built, furnishing power to various small units. If the wells are not too widely spaced, and the ground water is not far below the surface, each well may be connected to one general suction pipe in which the required suction is maintained by a single pump.

26. Centrifugal pumps are the dominant type in irrigation work. The power to be employed depends almost entirely on local conditions and the size of the plant. Electric power is probably the most satisfactory and economical, and is quite generally available; but, where coal is low in price, and the project is a large one, steam may be used Internal combustion engines are especially suited to individual installations of small size, and the Diesel type has often demonstrated its economy and reliability. It is now being used to a large and growing extent in

pumping. Waterwheels of various types, hydraulic rams, and windmills are also employed, mainly on small projects, where the work they are required to do is limited.

27. Employment of Ground Water in Irrigation.—The use, in irrigation, of ground water secured by pumping from wells has expanded rapidly. At first it was confined mainly to California, but the utilization of surface waters in other states has reached the point where ground waters are of great importance. Efforts to increase ground-water supplies are being made, and pumping has been employed to such an extent in many places as to lower the ground-water level. In several states the question of the title to ground waters is being considered, and it is generally conceded that title to the ground surface does not imply unlimited ownership in ground waters.

WATER STORAGE

RESERVOIRS

28. Advantage of Reservoirs.—Unless provision is made for storing water during the seasons when it is not needed for crop production, the entire supply of water cannot be considered available for irrigation. Under such circumstances, the important quantity is the run-off during the season of crop growth. However, if the flow during the remainder of the year is stored for use during the growing season, the annual run-off may be used as a basis in determining the capacity of the source of supply. For example, when storage is provided, the average annual precipitation on the northern Pacific coast is sufficient to produce good crops, but without storage the value of the rainfall for the purpose of irrigation is greatly lowered.

When ground water is used for irrigation, the water can be drawn only at a rather uniform rate and, since the daily yield from wells is seldom sufficient for the irrigation requirements for a considerable area, it may prove economical to provide a reservoir for storing the water pumped during the season when irrigation is not required and holding it for later use.

29. Functions of Reservoir in Irrigation Project.—Owing to the different conditions under which irrigation works and

water-supply systems are operated, there are some differences between the functions of a reservoir for irrigation purposes and one intended for a public water supply. The draft on the former varies considerably from year to year according to the wetness or the dryness of the seasons, whereas the consumption from the latter is fairly uniform. Also, irrigation reservoirs have to furnish large quantities of water during short periods of time and hence must be equipped with appliances for meeting these demands.

30. Capacity of Reservoir.—When the quantity of water required annually for irrigation has been estimated and the available supply from the stream or other source has been established, the next step is to compute the capacity of the storage reservoir. In determining this capacity, the demands that may be made on the irrigation system during the driest weather must be considered, and ample allowance must be made for losses due to evaporation and absorption or seepage. The method of making the computations is similar to that used in designing a reservoir for a city water supply.

During the growing season, the water will probably be used as fast as, or faster than, it is supplied. Hence, the capacity of the reservoir need only be sufficient to store the water that is required in excess of the flow during the growing season. However, the possible destructive effects of unusual storms must be considered. Even though the average rainfall is very small, there are occasional violent storms in which the annual average rainfall may be exceeded in a single day. Unless adequate provisions are made to take care of such storms, they may fill reservoirs to overflowing, destroy dams, and ruin entire irrigation systems.

31. Evaporation.—In the dry regions, where irrigation is principally practiced, loss of water due to evaporation is usually a serious matter. It is very difficult to estimate in advance the probable loss by evaporation, because experiments must be carried out under circumstances far different from those that will prevail when the reservoir is completed. Evaporation is greatest

in warm weather and during windy periods. It varies with the character of the reservoir; it is less in a deep reservoir than in a shallow one, and less in still than in running water. In most of the semi-arid regions it will range between 4 and 10 feet during the year, depending on local conditions.

The apparatus generally used for measuring evaporation is shown in Fig. 2 (*a*). It consists of a galvanized iron pan *a*, 36 inches square and 18 inches deep, which is provided with floats

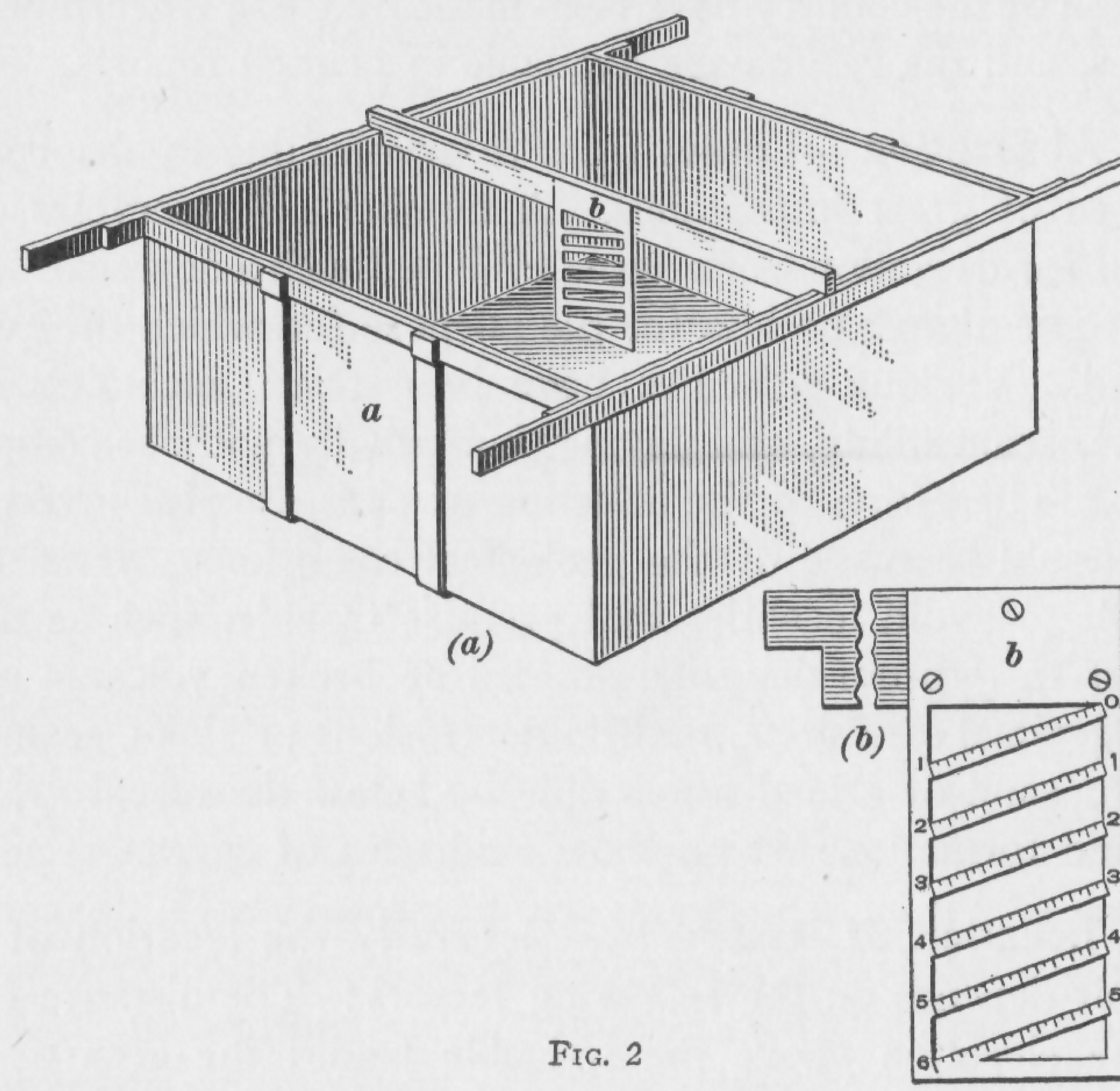

FIG. 2

of wood or hollow metal. The pan, nearly filled with water, is immersed in the body of water for which the evaporation is to be measured, and the drop in the level of the water surface, or the depth of water lost by evaporation, is determined by means of the scale *b*. The water within the pan is left 3 or 4 inches below the rim to prevent water from being slopped out of the pan by waves; also, the pan is so floated that its rim is several inches above the surrounding water, in order to prevent water from being washed into the pan. The scale *b* is divided into inclined sections, as shown in view (*b*), the vertical distance between two

consecutive numbered graduations being 1 inch. Hence, small vertical distances can be easily read.

While it is always desirable to make observations and investigations concerning evaporation for each particular project, it is not always possible to secure complete data from them. Valuable additional information may be obtained from the publications of the United States Geological Survey and the United States Weather Bureau. Elaborate observations covering a wide area of the country have been made by these Governmental agencies, and the results are available in printed reports.

32. Absorption.—Losses from reservoirs due to absorption are often of great importance. A number of reservoirs constructed for irrigation work have developed such a degree of leakage as to be almost useless for the purpose for which they were designed. Various remedies have been tried, including clay blankets, seam filling, and paving, but usually without complete success. Therefore, in the selection of sites, careful investigations should be made of the geological conditions. It is most desirable to avoid localities that promise trouble, such as those adjacent to gypsum deposits, seamed or broken volcanic rock, coarse-grained sandstone, natural depressions or sinks, seams or strata of sand or gravel which outcrop below the reservoir, and limestone formations which show evidences of caves.

33. Location of Reservoir.—Generally the location of the reservoir depends on the following factors: The distance from and the elevation above the irrigable lands; the area of the exposed water surface; and the topography and geology of the site. The altitude of the site must be sufficient to enable the water to flow by gravity to all or nearly all of the land to be irrigated. Also, it is always desirable to have the reservoir as near as possible to those lands. Not only are the length and cost of the canal and the water losses in transmission reduced, but the regulation of the water supply according to the needs of irrigation is made easier and less water is wasted. If possible, the water should be conducted the entire distance from the reservoir to the irrigated lands in an artificial conduit. In many cases, however, it is necessary to discharge the water into a natural

watercourse and divert it farther downstream. In such cases, the losses due to percolation and evaporation are greater than when an artificial channel of favorable cross-section can be used.

34. Survey of Site.—After the site for the reservoir has been selected, it should be surveyed carefully. A map should be constructed with a contour interval of not more than 10 feet, so that the capacity of the reservoir may be determined for any given height of dam; the scale may be 1,000 to 2,000 feet to the inch. It is usually desirable to run a traverse along the contour at the elevation of the top of the dam so as to obtain the area of the reservoir quite accurately. Stadia or plane-table surveys are suitable for topographic work.

A most careful survey of the actual dam site chosen is necessary. Borings should be made to determine the nature of the foundation and underlying material, and the geology of the site should be investigated by a geologist. From such surveys and examinations, it will be possible to proceed with the design of the dam, and to estimate the amount of materials required for its construction, and the approximate cost.

DAMS

35. Types of Dams.—Dams may be classified as diversion dams or storage dams, according to the task for which they are designed. Diversion dams are intended to raise the water for a few feet only, so as to divert part or all of the water into a conduit, no storage being contemplated. Storage dams are designed to retain the surplus flow, holding the water until it is needed.

Dams may be of timber, earth with or without a masonry core, rock-fill, concrete or other masonry, or metal. The type and details of construction and design depend almost entirely on local conditions. Where life may be endangered by failure of the dam, the most exhaustive investigations should be made to assure the safety of the dam.

36. Timber Dams.—For the smaller and less important dams, such as are often built on small irrigation works or by individuals, timber dams are often suitable. The timber crib

type is most generally used. Essentially, it consists of a series of cribs or enclosures 12 to 16 feet square, made of logs or timbers and filled with stone. The cribs are fastened to each other by timbers extending from each crib to the adjoining ones, the timbers being bolted or pinned together. The upstream faces of the cribs are covered with a tight plank sheeting, behind which is an earthen embankment. In favorable soils, the slope of this embankment may sometimes be as little as 1 vertical to 3 horizontal; but usually it will have to be about 1 to 5 or 6. The embankment should be rip-rapped, or paved with stones below the water level.

37. **Earth Dams.**—When constructed under favorable conditions, earth dams are cheapest for medium or even considerable heights. Essential features in an earth dam are as follows: The top of the dam should be far enough above the proposed water surface and the capacity of the spillway should be ample to prevent overtopping by the water, the excess height depending on the size of the reservoir, the storm flow of the stream, and the reach of the waves; the top width should be about one-fifth of the height plus 5 feet; both faces of the dam should have sufficiently flat slopes to prevent sloughing or sliding; a pavement or rip-rap should be provided on the up-stream face to prevent injury by wave action; there should be an impervious center core or up-stream face.

38. **Rock-Fill Dams.**—Where rock is plentiful and other construction material scarce, loose rock may be used in the body of the dam, other materials being employed for building a tight curtain on the up-stream face. Watertightness is usually secured by laying a covering of wood planks or steel plates, by placing a cement facing, or by dumping or sluicing earth into place. As in an earth dam, ample spillway capacity and proper slopes on both faces of the dam are safety essentials. Where conditions are favorable, this type of construction is both cheap and satisfactory.

39. **Masonry Dams.**—Most masonry dams now being built are of concrete; such dams are of two general types, gravity and

arch. The former depend for their stability entirely on their weight; the latter rely mainly on their arch action. Gravity dams are more common, but the majority of these are curved so as to secure some arch action, an extra degree of safety thus being provided. Hollow dams of reinforced concrete are also built; in these dams a thin, sloping up-stream floor, which may consist either of flat slabs or of a series of arches, is supported on buttresses.

40. **Movable Dams.**—Various types of movable dams, which are commonly of steel, are used generally for purposes of diversion or temporary storage. They are usually employed in connection with a sill or low dam in the stream, and a bridge or other structure to support the upper edge, although some types do not require the upper support.

TRANSPORTATION OF WATER

CONSTRUCTION OF WATERWAYS

CANALS

41. **Introduction.**—In order to make the stored water available for irrigation, it may be necessary to transport it for many miles. The amount of water to be transported, and the character of the country over which the line is to pass will usually determine the character of the conduit to be employed. Where the volume is large, and it is to be transported for a long distance and distributed in measured quantities over an extensive territory along the route, open canals are generally employed where possible, flumes or pipe lines being introduced when it is necessary to cross a deep valley. When the volume of water to be conveyed is not very great, flumes or pipe lines may be used for the entire length, in order to reduce the losses due to evaporation and percolation, which may be considerable in a canal.

42. **Comparison of Earthen and Lined Canals.**—Canals may either have earthen banks or be lined. Earthen canals are cheaper to construct but are often far from satisfactory because of trouble due to erosion, deterioration of the banks or channel,

loss of water, and cost of maintenance. As a consequence, despite their greater cost, lined canals have come into extensive use. In addition to preventing loss of valuable water, and trouble due to erosion and uncertain delivery of water, the lining allows greater velocity and a consequent reduction in the cross-section of the canal, with a saving in cost. In some cases, the water saved by a lined canal may be worth more than the extra cost of the lining. Concrete or cement mortar is the material now most generally used for lining, but other materials including lumber, clay, bituminous products, and stone are sometimes employed.

43. Concrete and Mortar Linings.—Concrete linings are usually from 2 to 4 inches in thickness and may or may not be reinforced. The mix should be 1:2:4 or 1:3:6, and the coarse aggregate should not exceed in size one-half of the thickness of the lining. In a number of places, thin linings composed of 1:3 mortar have been plastered on, or applied with the cement gun. They are cheaper than the thicker and more rigid linings, and have given excellent results under suitable conditions.

Concrete or other rigid linings should be placed only on well-settled ground. On new construction in fills, resulting settlement or shifting will crack or destroy the lining. Before a concrete or mortar lining is placed, the earth backing should be carefully prepared by rolling and moistening. The lining should be constructed in alternate sections with expansion joints at intervals of 20 to 40 feet, and the concrete or mortar should be well cured in order to protect it from too rapid drying.

44. Other Linings.—Puddled clay has been used as a lining and found effective in reducing seepage in many places. In a few canals where the irrigation water was clear and seepage losses were great, water containing silt was pumped into the canal and the silt was deposited in the proper places by checking the velocity of the water at those points.

Paving with stone is not uncommon. For good hydraulic properties, the stone should be well-graded and closely packed. Where a canal is constructed on a long fill that might settle

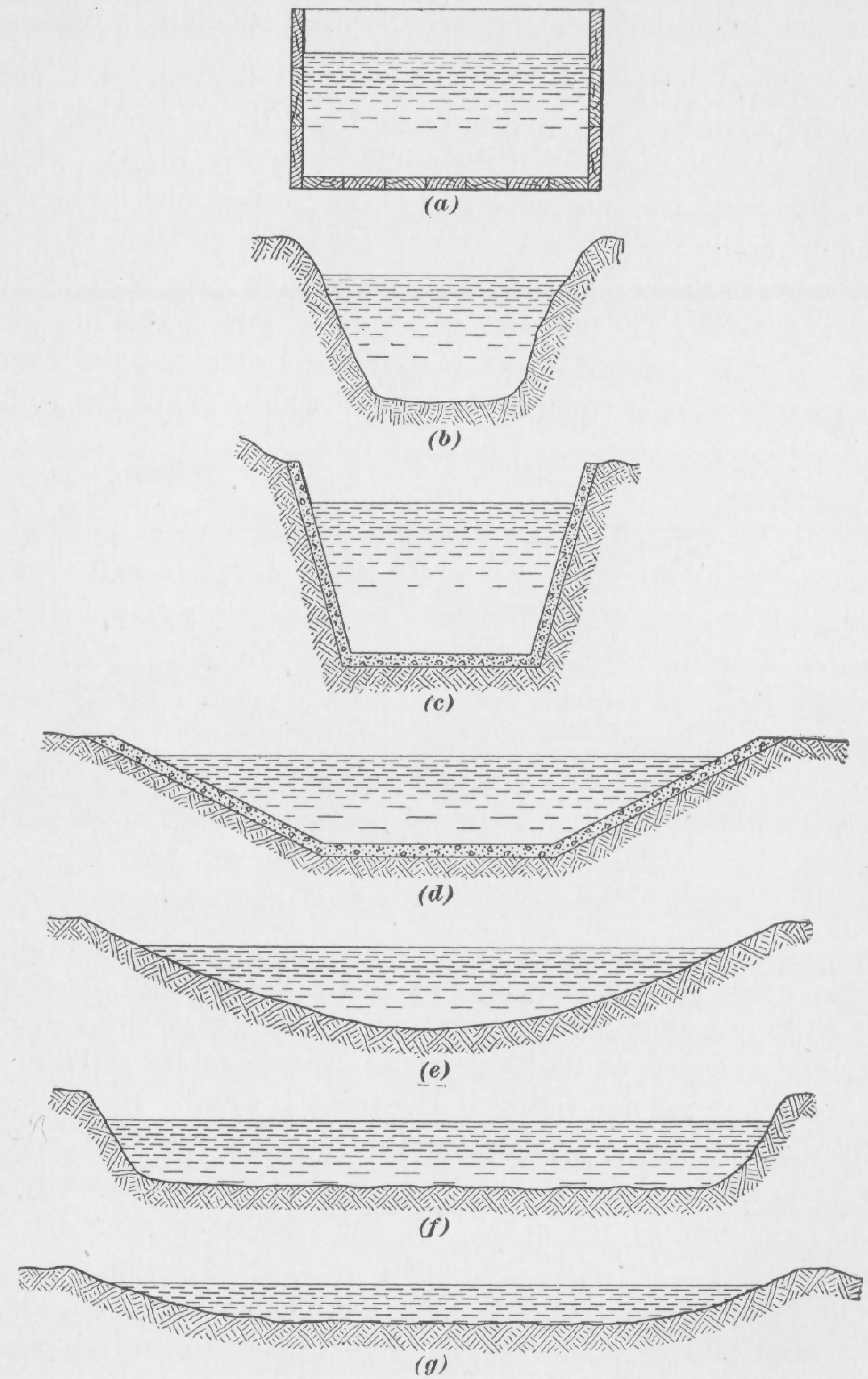

FIG. 3

unevenly and crack a concrete lining, it may be well first to place a wooden lining and later replace it with a concrete lining.

45. Canal Sections.—The most economic section for a canal from the standpoint of hydraulic efficiency is a rectangle with a depth equal to one-half of the width, as in Fig. 3 (*a*). However, this section is not widely used, as construction considerations generally make it advisable to use other sections, such as are shown in views (*b*) to (*g*). The sections usually employed are trapezoidal. The best section for general use is that in which the sides make an angle of 60° with the horizontal and the length of each side slope is equal to the bottom width. Then the width

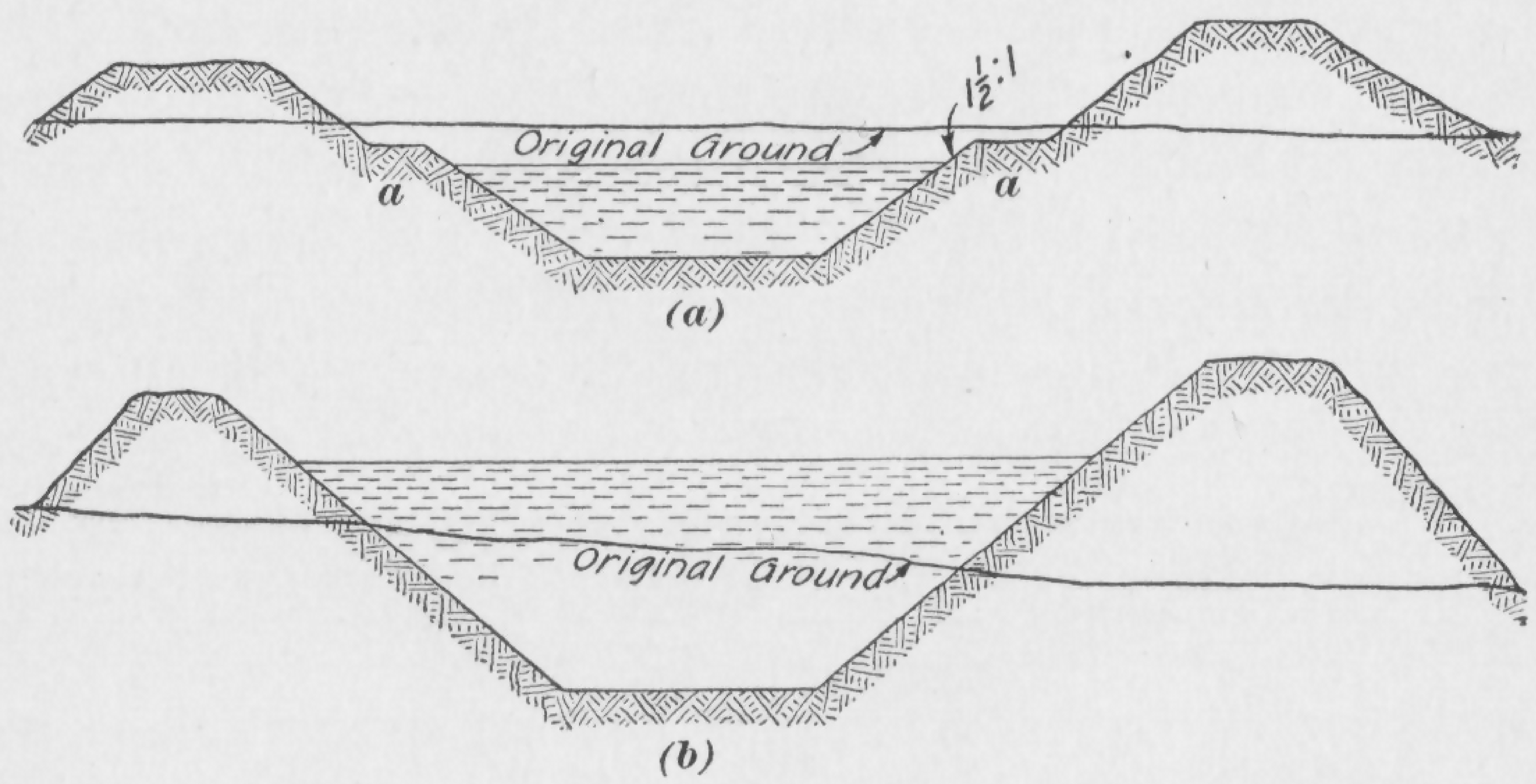

Fig. 4

of the water surface is twice that of the bottom and the depth is a little less than seven-eighths of the bottom width. However, the steepness of the bank slopes depends on the soil and on the material used for lining if the canal is lined. In ordinary soils, slopes of about $1\frac{1}{2}$ horizontal to 1 vertical are used. Also, the ratio of the depth to the width depends on the scarcity of water, the size of the canal, and the transverse slope of the ground.

On fairly level ground, a wide and shallow canal in which excavation and embankment are nearly equal is, for a given cross-sectional area, cheaper than a narrow and deep one, but requires a steeper grade to produce a given velocity and leads to greater water losses due to evaporation and seepage. When the canal is

constructed along a steep hillside, a narrow and deep channel may be necessary.

46. Typical Construction of Canals.—Canals are best built entirely in excavation, as indicated in Fig. 4 (*a*), berms *a* being provided in some cases and omitted in others. However, this construction is difficult and generally impossible because of the many changes in alinement necessary. Usually, a large portion of the canal is built partly in excavation and partly in embankment, as in view (*b*).

The thickness of the banks in fills should be ample to limit leakage and to eliminate danger of breaks. In cuts, a study of the soil is desirable to avoid seepage through a porous stratum.

FLUMES

47. General Features.—When it is desirable to carry water across a valley or ravine of moderate depth without making a long detour or having the water flow under pressure, a flume is generally used. It is a built-up open channel usually supported on a trestle. The materials generally employed for flumes are timber, iron, and steel, but concrete is also used.

The most common section for wood or concrete flumes is the rectangle and for metal flumes the half-circle. Wood-stave flumes are sometimes built with a semicircular shape.

48. Timber Flumes.—A simple form for a small timber flume, having a cross-section 4 feet by 2 feet 2 inches, is shown in Fig. 5. The dimensions of the timbers are given in the illustration. In this design, the bents may be 4 to 6 feet apart, and no mortising is needed, as all pieces can be assembled with spikes, bolts, and nails. A cross-section of a larger flume is shown in Fig. 6. Another type of wooden flume that is sometimes used is semicircular in cross-section and is constructed of wooden staves, which are supported by means of metal rods passing around them and through cross timbers, in the same manner as in the semicircular metal flume shown in Fig. 7.

In constructing flumes, special precautions should be taken to prevent spreading. In timber construction, sills are inserted at the bottom, and the sides are either braced with inclined struts,

as in Fig. 6, or held at the top with timber or wire ties connecting the sides. Well-seasoned timber should be used exclusively, and

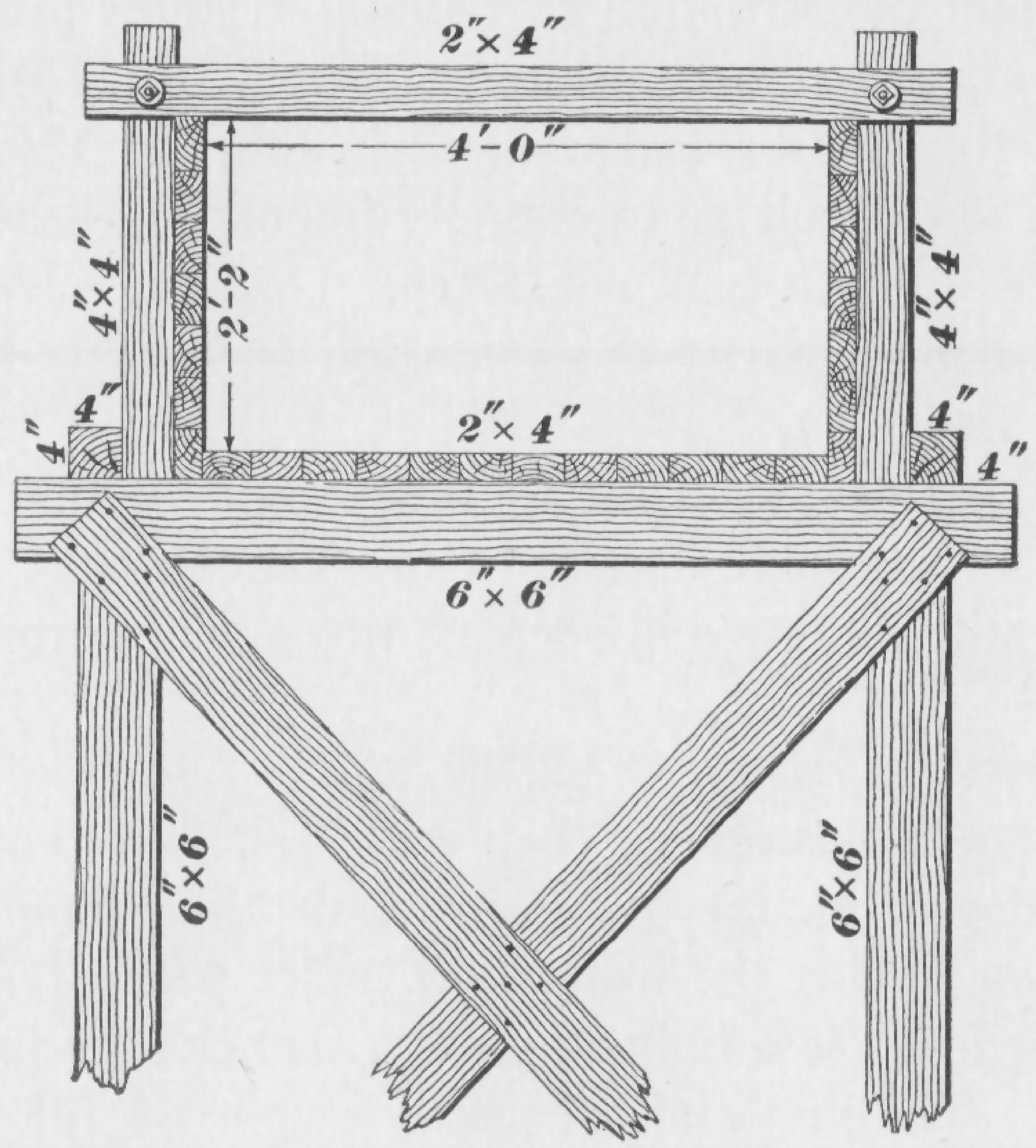

FIG. 5

the edges and inside faces should be planed. Some difficulty may be met in making the joints of the flume tight. Providing a small bead on the edge of the timber, coating the timber with paint or

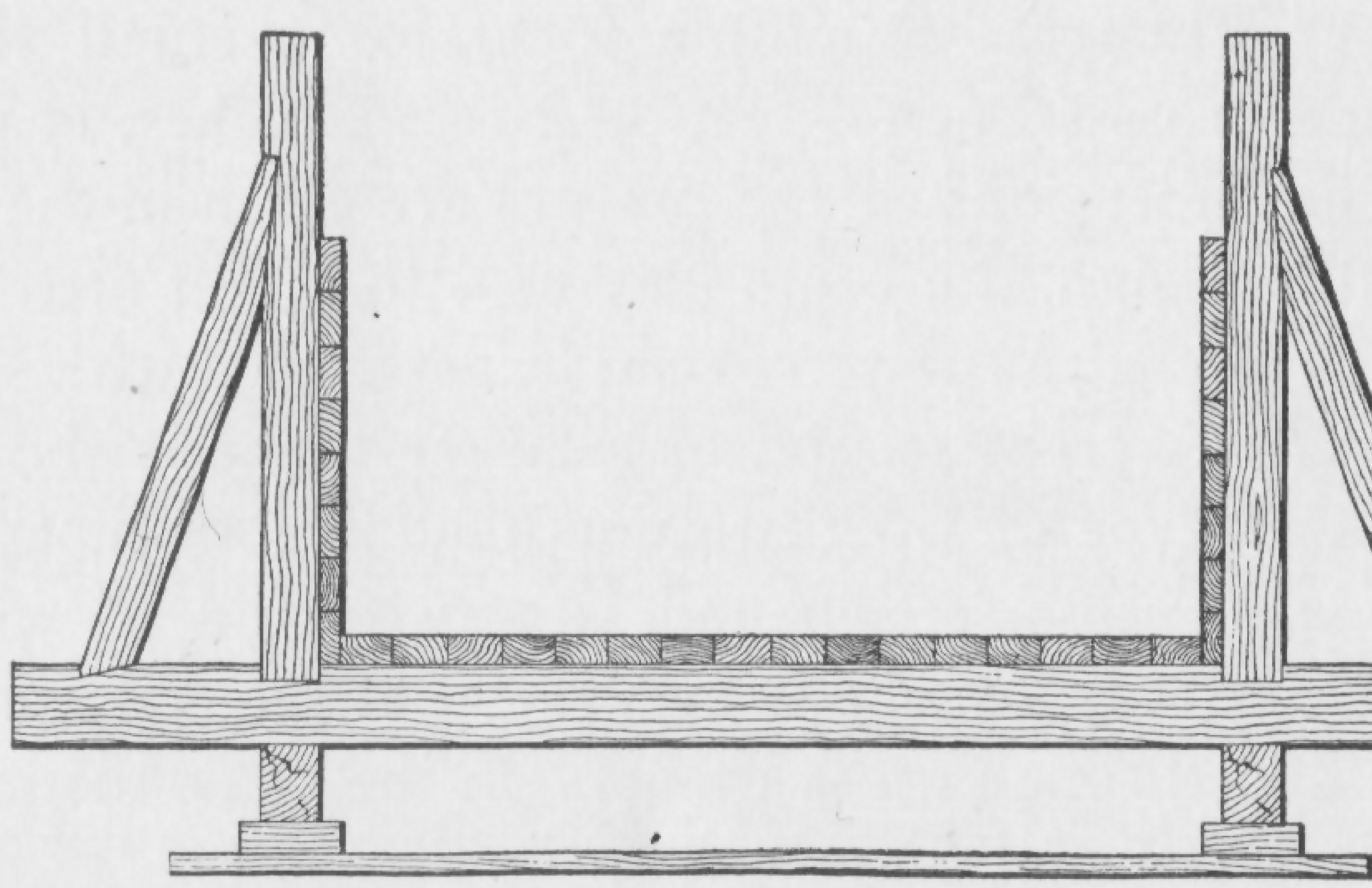

FIG. 6

asphalt, and calking the joints with oakum are among the standard methods of securing water-tightness. Special care should

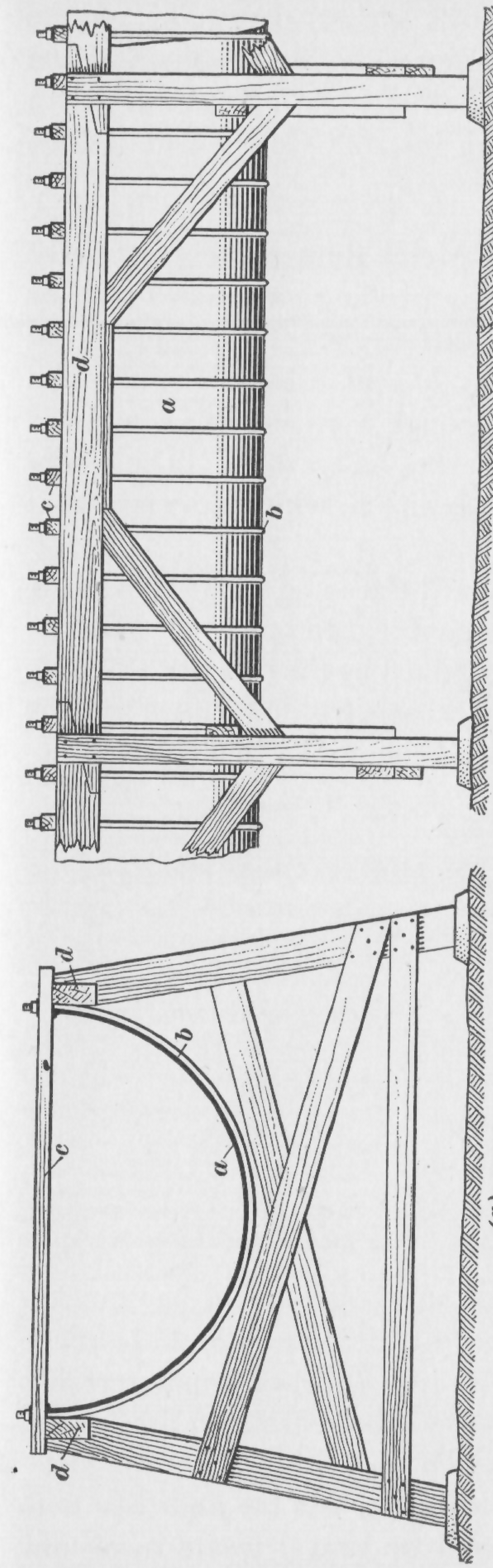

FIG. 7

be taken in placing the timbers of which the flume is made. These should be laid longitudinally, and in an even and smooth fashion in order to reduce the friction.

49. Concrete Flumes. When properly built, concrete flumes are of a permanent character and have a number of other advantages. The details of standard reinforced-concrete flumes constructed by the United States Reclamation Service are shown in Fig. 8, in elevation in view (*a*), in cross-section between supports in view (*b*), and in cross-section near a support in view (*c*). In each view, the reinforcement is represented by dot-and-dash lines.

The span *S* between columns *a* varies from 14 to 30 feet and the distance between the stiffeners *c*, which are required whenever the height *h* exceeds five times the thickness *t*, varies from 6 feet 8 inches to 9 feet 4 inches, according to the span. The depth *d* of the water varies from 1 to 4 feet, the width *b* from 4 to 12 feet, and the total

depth of the girders e from 9 to 21 inches, depending on the width b. One-third of the horizontal rods f in the floor of the flume are bent up at the quarter points when b is greater than 6 feet. The columns a are square in cross-section and rest on suitable footings.

50. Iron and Steel Flumes.—Metal flumes are frequently constructed, as shown in Fig. 7, by bending galvanized iron or steel sheets a to the form of a half-circle. These sheets are supported by means of iron rods b bent around them. The threaded ends of each rod pass through a cross-piece c and are held in place by means of nuts bearing on washers. The cross-pieces in turn rest on longitudinal beams d, which are supported by wooden trestle bents.

The sheets of metal are of standard sizes and the perimeter of the semicircular flume is made to conform to one of these sizes. The thickness of the sheets is determined by the size of the flume, the desirable thicknesses of steel sheets recommended by the United States Reclamation Service being given in Table I.

TABLE I

THICKNESS OF STEEL SHEETS FOR SEMICIRCULAR FLUMES

Length of Semicircle Inches	Gage of Metal
Up to 48	22
48 to 96	20
96 to 132	18
132 to 168	16
168 to 204	14
204 to 252	12

To secure the best flow, rivets and bolts should be countersunk on the inside, and the interior of the flume should be made as smooth as possible. Wrought iron of good quality resists rusting better than steel and its use is increasing in the United States. Protection against corrosion is desirable, and the metal may be painted first with a coat of water-gas tar and then with a coat of coal-tar pitch. As sand or gravel wears away any

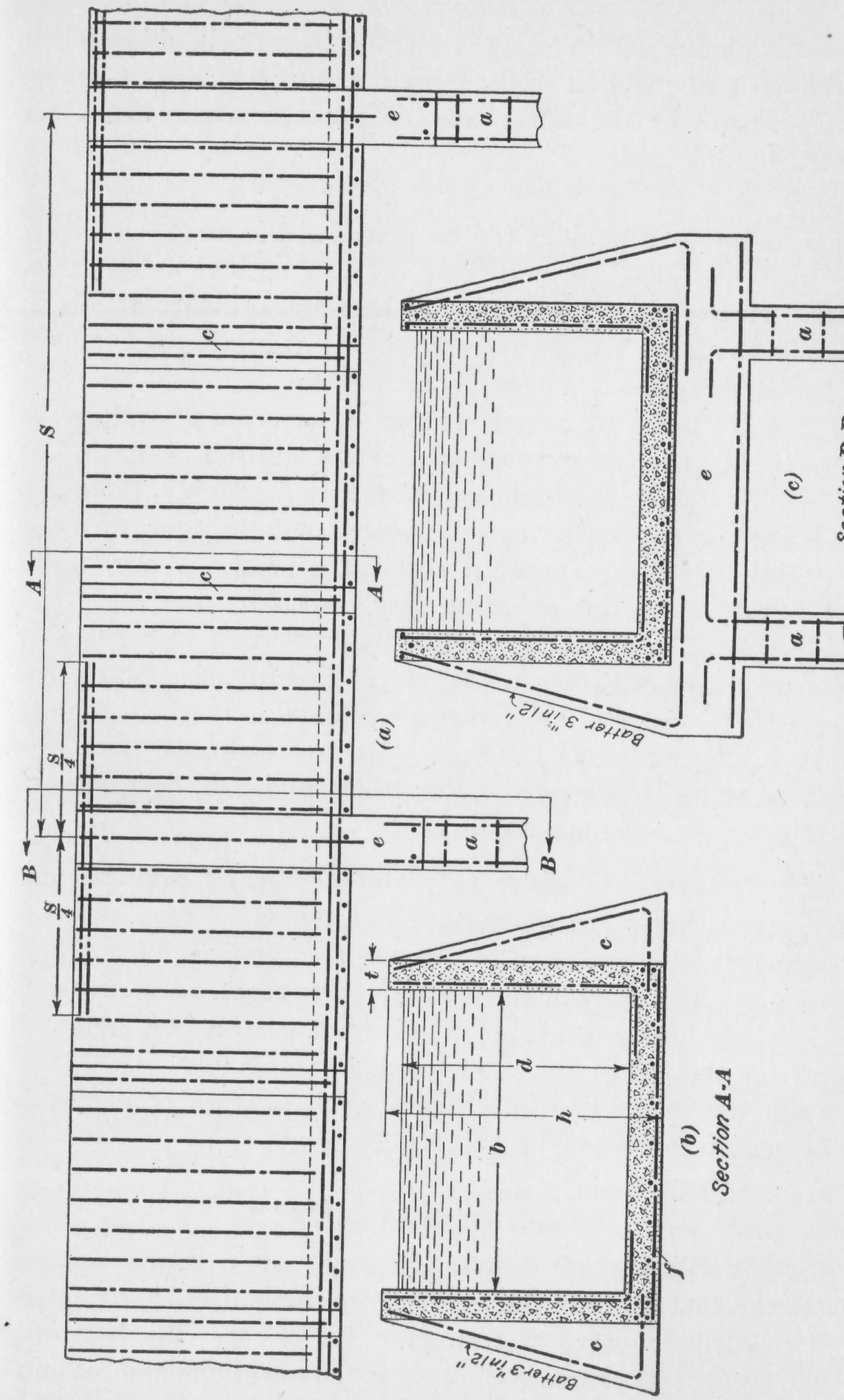

Fig. 8

protective lining within a short time, special care should be taken to remove such material from the water before it enters the flume. If the length of the flume exceeds 50 feet, expansion joints should be provided.

PIPES AND TUNNELS

51. Pipes.—Pipe lines can be placed either above or below the hydraulic gradient, and therefore can be laid along direct lines. However, it is advisable to keep the pressure as low as possible to reduce leakage and the number of breaks, and to have as few depressions and summits as possible.

Pipes are more advantageous than other types of waterways because they are less exposed to accident and less water is lost in them by leakage, seepage, and evaporation. Hence, there is a tendency to more extensive use of pipes for conveyance of water in irrigation, except where the cost is too great. Where water has a great value, this objection may not hold.

Pipes may be of wood, steel, wrought or cast iron, plain or reinforced concrete, or vitrified clay. Reinforced concrete has been extensively used in irrigation work and is very satisfactory under heads as high as 110 feet. For greater heads, steel, cast iron, or wood is employed. For heads under 20 feet, clay or plain-concrete pipes are used.

52. Tunnels.—In order to avoid detours it may become necessary to carry the water through a tunnel. Experience has shown that it is desirable to provide the tunnel with a lining, even when it runs through rock. In some cases, timbering only may be used; but a concrete lining is most satisfactory. An arch form for the top of the tunnel resists caving best. Generally, the smallest tunnel in which it is possible to work economically is about 4 feet wide and 6 feet high.

DESIGN OF WATERWAYS

53. Preliminary Surveys.—Surveys of a more or less extensive character are the necessary preliminaries to the design and construction of a waterway. This is especially true of a canal, because its course is confined to narrow limits and the location is a matter of considerable importance.

Alternative general locations for the waterway often must be considered. On the one hand, there may be costly construction with the ability to supply all the land by gravity, and on the other hand, cheaper construction with the necessity for pumping the water to a portion of the land to be irrigated. Usually, several possible routes will have to be surveyed. Frequently it will pay to make a topographic map of the entire area, using a plane table or stadia. Study of such a map may indicate entirely new possibilities, or show where pipe or tunnel sections may be combined with flumes or canals to obtain a cheaper construction.

54. Grade for Canal.—In locating a canal, it is necessary to consider the quantity of the water available as compared with the demand, as well as the topography of the country. If the amount of water is ample, the canal may be given the minimum grade to produce a satisfactory velocity, so that the greatest possible area of land may be irrigated by gravity. But if the amount of water is not sufficient to irrigate all the land, the grade should be selected so as to give the water as great velocity as possible, in order to reduce the amount of water lost through evaporation and seepage. Roughly speaking, the seepage from a canal is proportional to the wetted area, and a small cross-section is desirable where it is practical. Where there is plenty of water, more attention can be given to economy of construction.

55. Desirable Velocity of Flow in Canal.—The minimum grade desirable for a canal is one that will maintain a velocity great enough to prevent the growth of plants and weeds and lessen the deposition of silt. On the other hand, in an earthen canal too great a velocity will result in bank cutting and erosion. The maximum permissible velocity depends on the type of soil, but it should rarely exceed 3 feet per second. This is sufficient to prevent vegetable growth and excessive accumulation of sediment and is safe except in very light soils. In lined canals, much higher velocities are permissible, and such channels are frequently designed for velocities as high as 10 or 15 feet per second.

56. Alinement of Canal.—Straightness is not a very important factor in canals for irrigation purposes, but the more direct

the line the less the cost of construction will be, provided cuts and fills are not excessively heavy and balance to a reasonable degree. Also, the losses from evaporation and absorption will be less in a shorter line.

Sharp curves are objectionable because of the danger of erosion and the reduction in the velocity of flow. In general, a radius of curvature of 3 to 5 times the top width of the canal is suitable; with high velocities the radius of curvature should be increased, and on sharp curves the grade of the canal should be steeper to maintain the desired velocity.

57. Required Capacity.—The required carrying capacity of a waterway will depend on the area of the land to be served, the duty of the water, and the losses of water in transit. An additional allowance should be made in the size of the waterway to compensate for the losses in carrying capacity due to silting or other causes, and as a factor of safety. Thus, where the maximum estimated requirement of water for a 15-day period is $\frac{1}{3}$ foot over the entire area, and the total area to be irrigated is 6,000 acres, the water required for irrigation will amount to about $\frac{\frac{1}{3}\times 6{,}000}{15}=133$ acre-feet per day or $\frac{133\times 43{,}560}{86{,}400}=67$ second-feet; and if the seepage and other losses are 25 per cent. of the flow, provision must be made for an actual flow of about 90 second-feet. To this should be added an allowance of 5 per cent. or more for silting and depreciation of hydraulic qualities, these factors depending on local conditions.

58. Flow in Open Channels.—In most countries, open channels are designed by the use of the following formulas:

$$c=\frac{\frac{1.811}{n}+41.65+\frac{.00281}{s}}{1+\frac{n}{\sqrt{r}}\left(41.65+\frac{.00281}{s}\right)} \qquad (1)$$

$$v=c\sqrt{rs} \qquad (2)$$

$$Q=av \qquad (3)$$

in which c and n are coefficients;
s = rate of grade of channel;
r = hydraulic radius of channel, in feet;
v = velocity of flow, in feet per second;
Q = discharge, in second-feet;
a = cross-sectional area of channel, in square feet.

Formula 1 is Kutter's formula and formula 2 is Chezy's formula. The hydraulic radius of a section is found by dividing the area of the cross-section of the flowing water, in square feet, by the wetted perimeter of the cross-section, in feet.

59. Values of n.—The selection of the proper value of n is very important. The following values are based on experiments by Scobey, the results of which were published by the United States Department of Agriculture, and are applicable to velocities up to about 5 feet per second and to hydraulic radii up to about 2 feet. Where the velocity or hydraulic radius greatly exceeds the respective limit, a lower value of n may be used.

For earth channels n varies from .016 to .030. The lowest value can be used where the natural material of which the canal is composed becomes slick when wet or a slick deposit of silt accumulates, the edges are free from vegetation, the water is free from moss or other aquatic growths, and the alinement is good. A value of .020 is recommended for well-constructed canals in firm earth or fine packed gravel, with good alinement, and banks clean-cut and free from disturbing vegetation. For canals where the retarding influences of moss, grass near the edges, or scattered cobbles begin to show, n should be .025. Where maintenance is neglected, values begin at this figure and increase rapidly; .025 is a good value to use in the design of small head ditches, or small ditches to serve one or two farms. If the canal is subject to heavy growths of moss or other aquatic plants, and has irregular banks and a rough bottom, a value of .030 may be used. For values of n above .030, the channel is much choked with vegetation, very irregular, overhung with dragging trees or grasses, and in poor condition.

60. For the highest grade of material and workmanship, with the lining in exceptionally good condition, the value of n

for concrete-lined channels is .012. For this value of n, the surface of the lining should be as smooth as a troweled sidewalk; the water must be free of shifting material; the alinement must consist of long tangents with spiraled curves; and the interior of the channel must be of true and uniform section. Other values of n for channels lined with concrete are as follows: for the ordinary curvature of channels built in canyons, .013; for usual good construction on moderate-size channels, .014, and, if the curves are sharp, .015; for rough linings, with uneven joints, .017; and, for very rough concrete, with sharp curves and deposits of gravel and moss, irregular cross-section, etc., .018. In masonry-lined canals, values of n approximating those for average concrete construction may be used. Where graded and well-packed cobblestones are used for lining, n may be taken as .027.

61. For wooden flumes, the value of n is .012 for the best construction in which surfaced lumber running longitudinally is used, and where there are long tangents and gentle curves; .013 for flumes having considerable curvature; .015 for flumes of unplaned lumber; and .016 for rough construction with sharp bends.

Metal flumes have a value of .012 for countersunk joints and smooth construction, .015 for joints that project into the water section, and .022 for material having corrugations at right angles to the axis of the flume.

62. Approximate Formulas for Flow.—The following approximate formulas may be used for canals with earth banks in good condition:

$$v=\sqrt{\frac{100{,}000r^2s}{9r+35}} \qquad (1)$$

$$s=\frac{(9r+35)v^2}{100{,}000r^2} \qquad (2)$$

For wooden flumes or masonry-lined channels, approximate formulas are

$$v=\sqrt{\frac{100{,}000r^2s}{6.6r+.46}} \qquad (3)$$

$$s=\frac{(6.6r+.46)v^2}{100{,}000r^2} \qquad (4)$$

63. Illustrative Examples.—In designing waterways, it is important to remember that the value of c obtained by Kutter's formula is not affected appreciably by even a fairly large change in the rate of grade s. Hence, if the velocity corresponding to a rate of slope s_1 is v_1, the velocity v corresponding to another rate of slope s may be found closely by the relation

$$v=v_1\sqrt{\frac{s}{s_1}} \qquad (1)$$

and the rate of slope s for a velocity v may be found by the relation

$$s=\frac{s_1v^2}{v_1^2} \qquad (2)$$

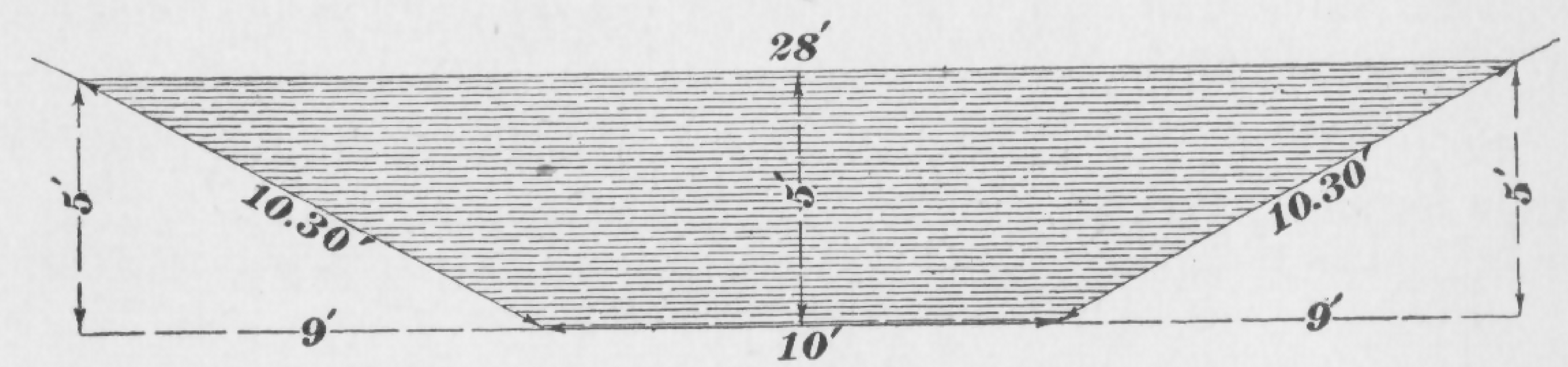

FIG. 9

Also, for a given shape of section, the value of c is not altered much by a change in the dimensions. Therefore, if v_1 is the velocity for a hydraulic radius r_1, the velocity v corresponding to a radius r is

$$v=v_1\sqrt{\frac{r}{r_1}} \qquad (3)$$

The following typical examples illustrate the methods of designing waterways for irrigation projects:

EXAMPLE 1.—If the fall in an earthen canal having the cross-section shown in Fig. 9 is 5.25 feet per mile, determine the mean velocity of flow (*a*) by the approximate formula and (*b*) by the formulas in Art. 58.

SOLUTION.—(*a*) Here, $s=\frac{5.25}{5{,}280}=.000994$, the area of the water section

is $\frac{10+28}{2}\times 5=95$ sq. ft., the wetted perimeter is $10+2\times 10.30=30.6$ ft., and $r=95\div 30.6=3.1$ ft. Then, by formula 1 of the preceding article,

$$v=\sqrt{\frac{100{,}000r^2s}{9r+35}}=\sqrt{\frac{100{,}000\times 3.1^2\times .000994}{9\times 3.1+35}}=3.9 \text{ ft. per sec. Ans.}$$

(*b*) If the value of n in formula 1, Art. 58, is taken as .025,

$$c=\frac{\frac{1.811}{.025}+41.65+\frac{.00281}{.000994}}{1+\frac{.025}{\sqrt{3.1}}\times\left(41.65+\frac{.00281}{.000994}\right)}=71.7$$

Then, by formula 2,

$$v=c\sqrt{rs}=71.7\sqrt{3.1\times .000994}=3.98 \text{ ft. per sec. Ans.}$$

EXAMPLE 2.—The required capacity of a wooden flume of rectangular cross-section is 250 second-feet. The width of the flume is not to exceed 8 feet and the velocity of flow is to be about 6 feet per second. If the best construction is to be used, what should be (*a*) the depth of the water and (*b*) the rate of grade of the flume?

SOLUTION.—(*a*) The required cross-sectional area of the flume is, from formula 3, Art. 58,

$$a=\frac{Q}{v}=\frac{250}{6}=41.7 \text{ sq. ft.}$$

and the depth, which is found by dividing this area by the width, is $41.7\div 8=5.21$ ft., say 5 ft. 3 in. Ans.

(*b*) First the approximate rate of grade is found by formula 4 of the preceding article. In this case $r=\frac{8\times 5.25}{8+2\times 5.25}=2.27$ ft. and

$$s=\frac{(6.6r+.46)v^2}{100{,}000r^2}=\frac{(6.6\times 2.27+.46)\times 6^2}{100{,}000\times 2.27^2}=.00108$$

This result should be verified by applying formulas 1 and 2, Art. 58, n being taken as .012. Then, c is found to be 144 and

$$v=c\sqrt{rs}=144\sqrt{2.27\times .00108}=7.15 \text{ ft. per sec.}$$

The more accurate rate of slope can now be found by formula 2 of this article, in which $s_1=.00108$, $v=6$ ft. per sec., and $v_1=7.15$ ft. per sec. Thus,

$$s=\frac{s_1v^2}{v_1^2}=\frac{.00108\times 6^2}{7.15^2}=.000761. \text{ Ans.}$$

EXAMPLE 3.—An earthen canal having a trapezoidal cross-section and side slopes of $1\frac{1}{2}$ horizontal to 1 vertical is to discharge 250 second-feet. If the desired depth of water is 6 feet and the desired velocity of flow 2 feet per second, determine (*a*) the bottom width and (*b*) the top width.

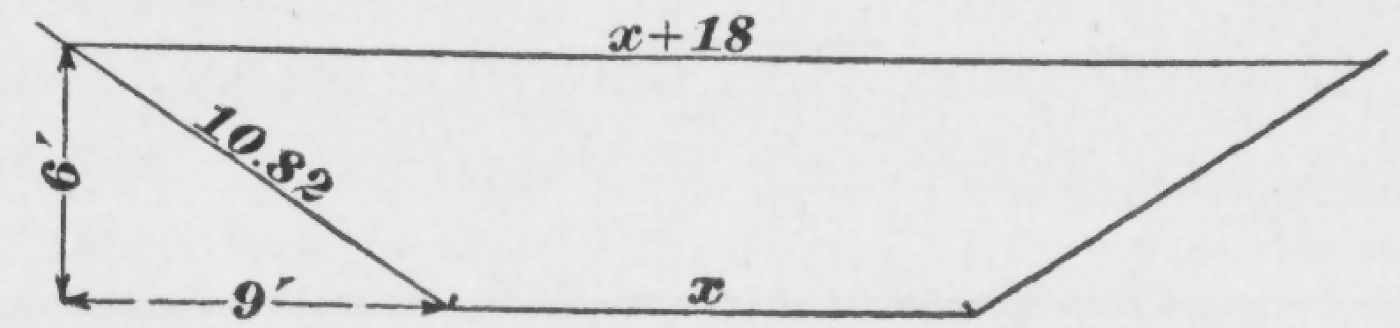

FIG. 10

SOLUTION.—(*a*) The required cross-sectional area of the canal is $250\div 2=125$ sq. ft. If the bottom width of the canal is represented by x, the dimensions of the section are as shown in Fig. 10 and the area is $\frac{x+x+18}{2}\times 6=6x+54$. Hence, $6x+54=125$ and $x=11.83$ ft. or 11 ft. 10 in. Ans.

(*b*) The top width is 11 ft. 10 in.+18 ft.=29 ft. 10 in. Ans.

EXAMPLE 4.—A concrete-lined canal for which $n=.013$ is to discharge 400 second-feet when laid on a grade of .0015. If the cross-section is to be a trapezoid in which the top width is equal to twice the bottom width and the side slopes are to make an angle of 60° with the horizontal, what should be the bottom width?

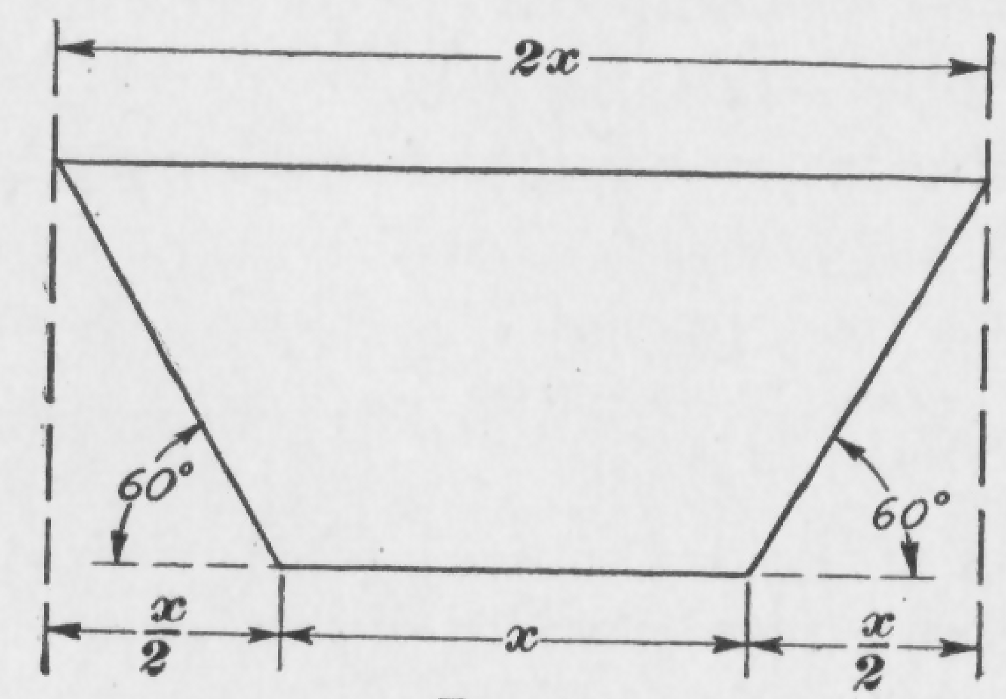

FIG. 11

SOLUTION.—If the bottom width is denoted by x, as in Fig. 11, the top width is $2x$, the horizontal projection of each side slope is $\frac{2x-x}{2}=\frac{x}{2}$, the depth of the canal is $\frac{x}{2}\tan 60°=.866x$, and the length of each slope is $\frac{x}{2}\sec 60°=x$. Then, the area of the section $a=\frac{x+2x}{2}\times .866x=1.299x^2$, the wetted perimeter $p=x+2x=3x$, the hydraulic radius $r=1.299x^2\div 3x=.433x$, and the required velocity $v=\frac{Q}{a}=\frac{400}{1.299x^2}$. If these values of

v and r are substituted in formula 3, Art. 62, the resulting equation cannot be readily solved for x. Hence, it is easier to assume values of x and determine the corresponding values of the discharge Q.

For $x=7$ ft., $r=.433\times7=3.03$ ft., and by Kutter's formula, $c=138$. Then, by formula 2, Art. 58,

$$v=c\sqrt{rs}=138\sqrt{3.03\times.0015}=9.3 \text{ ft. per sec.}$$

and, by formula 3, Art. 58,

$$Q=av=1.299\times7^2\times9.3=592 \text{ sec.-ft.}$$

As this discharge exceeds the given quantity, a smaller value of x may be assumed and the calculations repeated. However, after one assumption has already been made, the value of x can be determined directly. If x_1 is the assumed bottom width and x is the required value, then since r is proportional to x, $\frac{x}{x_1}$ may be substituted for $\frac{r}{r_1}$ in formula 3, Art. 63. Also, if a_1 is the area corresponding to x_1 and a the area corresponding to x, then $\frac{a}{a_1}=\frac{1.299x^2}{1.299x_1^2}$ or $a=\frac{a_1x^2}{x_1^2}$. Therefore, if Q_1 is the discharge corresponding to x_1 and Q that corresponding to x,

$$\frac{Q}{Q_1}=\frac{av}{a_1v_1}=\frac{\frac{a_1x^2}{x_1^2}\times v_1\sqrt{\frac{x}{x_1}}}{a_1v_1}=\sqrt{\frac{x^5}{x_1^5}} \text{ or } x=x_1\sqrt[5]{\frac{Q^2}{Q_1^2}}$$

Hence, the correct value of x can be found closely by this relation in which $x_1=7$, $Q=400$, and $Q_1=592$. Thus,

$$x=7\sqrt[5]{\frac{400^2}{592^2}}=5.98, \text{ say 6 ft.}$$

As a check, for $x=6$ ft., $r=2.6$ ft., $c=135$, $v=8.44$ ft. per sec., and $Q=395$ sec.-ft. Hence, a bottom width of 6 ft. or a little greater would be used. Ans.

EXAMPLES FOR PRACTICE

1. A rectangular timber flume is 10 feet wide and the depth of the water in it is 5 feet. If the fall is 9 inches per mile, what is the velocity of flow by the approximate formula? Ans. 2.29 ft. per sec.

2. (*a*) Find the velocity in the preceding example by the formula of Art. 58 if n is taken as .015. (*b*) Compute the discharge in second-feet.

Ans. $\begin{cases}(a) \text{ 2.17 ft. per sec.}\\(b) \text{ 108 5 sec.-ft.}\end{cases}$

3. A concrete-lined canal, which is to discharge 500 second-feet, is to have a trapezoidal cross-section, the top width being twice the bottom width and the depth of the water seven-eighths of the bottom width. If the velocity of flow is to be 10 feet per second, what should be the bottom width? Ans. 6.17 ft.

4. If the value of n for the canal in example 3 is .013, determine the required rate of slope. Ans. .00201

5. An earthen canal, which is to be laid on a grade of .002 and so constructed that n may be taken as .025, is to discharge 150 second-feet. If the cross-section is to be a trapezoid in which the top width is three times the bottom width and the side slopes are 2 horizontal to 1 vertical, what is the required bottom width? Ans. 6.18 ft.

LATERALS

64. Construction of Laterals.—In planning an irrigation system, a decision must be made as to the provision of laterals and sublaterals for delivering the water to the individual farms. The water may be delivered only out of the main canal, or main laterals and sublaterals leading to the small parcels of land may be built, or some intermediate method may be adopted. On many of the older systems, only the main canal and the main laterals were constructed as part of the enterprise, the water users being required to build the sublaterals to their farms; in some cases, even the main laterals were built by the farmers. Since the sublaterals often had to be many miles in length, the cost to the farmer of providing means for bringing the water of the irrigation system to his land was very heavy. Moreover, when this plan is adopted, there is likelihood that the ditches will be laid out poorly and will have inadequate capacity, since the first farmers will not wish to increase their investment by constructing a ditch large enough to provide for future arrivals. There is, however, assurance that the ditches will not be built until they are needed, and the smaller cash outlay in the early stages of the project may be desirable.

On the other hand, many irrigation projects have included the construction of a complete system of laterals and sublaterals with the idea that development would thus be stimulated. Such a practice often results in high depreciation before the laterals are used, and in heavy maintenance costs. Therefore, it is usually desirable to defer the construction of the smaller sublaterals and structures until there is a demand for them. The more general

TABLE II

CARRYING CAPACITY OF FARM DITCHES

Dimensions			Grade			Mean Velocity in Feet per Second	Carrying Capacity		
								Miner's Inches	
Bottom Width Inches	Depth Inches	Side Slope	Inches per Rod	Feet per 100 Feet	Feet per Mile		Cubic Feet per Second	Under 6-Inch Pressure	Under 4-Inch Pressure
			¼	.126	6.65	.80	.83	33	41
			½	.252	13.30	1.17	1.22	49	61
			¾	.379	20.00	1.41	1.46	58	73
			1	.505	26.65	1.61	1.67	67	84
			1¼	.631	33.30	1.80	1.87	75	94
16	6	1½:1	1½	.758	40.05	1.99	2.07	83	104
			1¾	.884	46.70	2.17	2.26	90	113
			2	1.008	53.50	2.32	2.41	96	120
			2½	1.260	66.60	2.60	2.70	108	135
			¼	.126	6.65	1.14	2.67	107	133
			½	.252	13.30	1.61	3.77	150	187
			¾	.379	20.00	1.99	4.65	186	232
			1	.505	26.65	2.30	5.38	215	269
			1¼	.631	33.30	2.57	6.02	241	301
24	9	1½:1	1½	.758	40.05	2.82	6.60	264	330
			1¾	.884	46.70	3.04	7.12	285	356
			2	1.008	53.30	3.24	7.58	308	379
			2½	1.260	66.60	3.60	8.43	367	422
			¼	.126	6.65	1.43	7.15	286	357
			½	.252	13.30	2.03	10.15	406	508
			¾	.379	20.00	2.52	12.60	504	630
			1	.505	26.65	2.89	14.45	578	722
			1¼	.631	33.30	3.22	16.10	644	805
36	12	2:1	1½	.758	40.05	3.54	17.70	708	885
			1¾	.884	46.70	3.84	19.20	768	960
			2	1.008	53.30	4.13	20.65	826	1,032
			2½	1.260	66.60	4.62	23.10	924	1,155

practice at present is to construct distributing systems to serve units of comparatively small size or to reach within a reasonable distance, as ¼ or ½ mile, from each subdivision of the project.

65. Required Capacity of Laterals.—The quantity of water to be carried by a lateral is determined mainly by the area of the district supplied by the lateral, although it is also affected by other factors. Obviously, a larger lateral is required for a large district than for a small one; but the size of the lateral is not in direct proportion to the area of the district, because the ratio of the maximum rate of water consumption to the average rate is likely to be greater for a small tract than for a large one. When the area to be irrigated does not exceed 100 acres, the discharge of the lateral should be at least 10 second-feet. For areas between 100 and 3,600 acres, a good rule is to assume that the required discharge of the lateral, in second-feet, is equal to the square root of the area to be irrigated, in acres, or

$$Q=\sqrt{A}$$

in which Q = required discharge of lateral, in second-feet;
A = area to be irrigated, in acres.

For larger areas, the discharge should be not less than 1 second-foot for each 60 acres to be supplied.

Where the irrigation water carries a large amount of material in suspension, which is likely to settle out while the water is flowing through the lateral, the capacity of the lateral when constructed should be considerably in excess of the required discharge. Then, the lateral can be kept in service during the entire irrigating season and cleaning can be done in the slack period while the water is shut off.

66. Maintenance of Laterals.—Ditches must be kept clean, for neglect results in decreased carrying capacity. Maintenance of laterals by community efforts is not generally desirable because of the difficulty of making each one do his share of the work; but local maintenance is advantageous, as a rule, since the cost will be less than with imported or organized labor. It is usually the most effective plan to place the responsibility on the management, but to use local labor under the direction of a foreman responsible only to the irrigation project.

67. Farm Ditches.—Farm ditches, or field ditches, are the final link in bringing the water to the fields, where it is actually

applied to the land. These should not lead from the main canal, which should have as few openings in the banks as possible, but should draw water from the laterals or sublaterals. The capacity required for the farm ditches depends on the manner of delivery of the water, the method of applying it, the duty of the water, the size of the farm, the kind of crop, the nature of the soil, and other factors. The delivery from the ditch varies from 1 to 2 second-feet up to as much, at times, as 8 or 10 second-feet. Three common forms of farm ditches are shown in Fig. 12. In fine

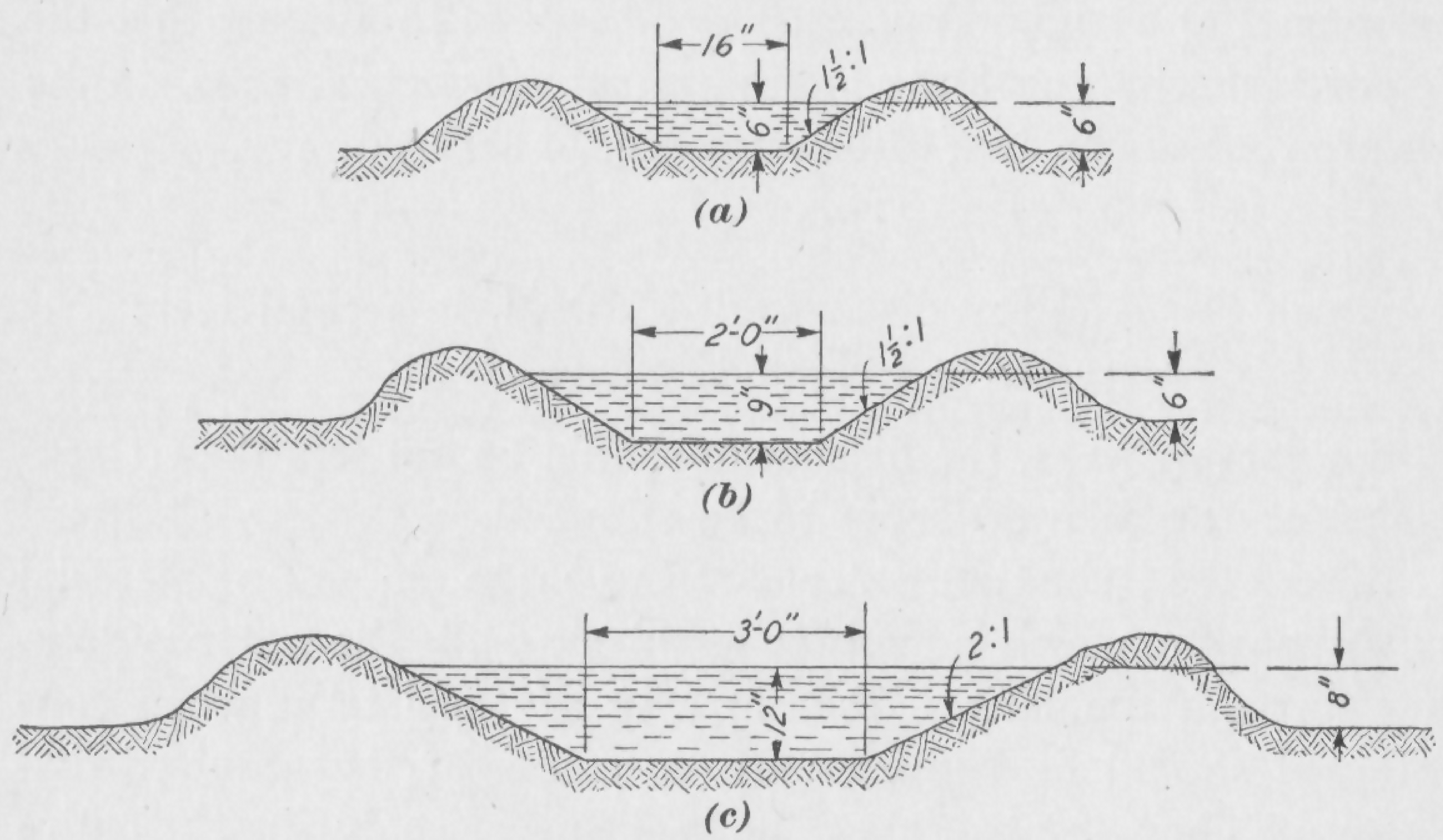

FIG. 12

sand, the grade should be such that a velocity of about 1 foot per second will be attained; on more suitable soils, velocities of 2 or even 3 feet per second may be safe and satisfactory. Carrying capacities of these ditches on various grades are shown in Table II.

Farm ditches should be located with foresight and consideration. Sufficient water should be secured to irrigate the entire area. The water should be conveyed from the canal or other source of supply to the highest point on the farm by means of laterals or sublaterals, and thence distributed to the various subdivisions by means of farm ditches. It is undesirable to build ditches for the lower part of a farm, and later be obliged to build a second series of ditches for the higher land.

CANAL STRUCTURES

68. Headworks.—At the point where the water from a stream is to be diverted into a canal, a group of structures known as headworks must be built. For the purpose of forcing part of the water into the canal, a diversion weir or dam is erected across the stream; also, in order that the quantity of water admitted to the canal may be under control at all times, a regulator with properly designed gates is built in the canal head. The headworks should preferably be located where the banks of the stream, especially the bank in which the regulator is built, are of material that cannot be easily eroded; this precaution is taken to guard against the possibility of the headworks being washed out or of the stream changing its course so as to render the headworks useless. Furthermore, the elevation of the headworks should be such as to permit the water to flow by gravity with a proper velocity to the land that is to be irrigated.

Provisions should be made to prevent the entrance of sediment that is likely to clog the canal. This may be accomplished by arranging to have the water enter the canal in a thin sheet over the top of a long weir and constructing the regulator parallel to the current in the stream so that sand and gravel will be sluiced away from the entrance to the canal. In spite of these precautions, silt may collect in the canal and it is advisable to provide, at the end of the diversion weir adjacent to the entrance to the canal, gates whose sills are at the level of the bed of the stream and which can be opened quickly from the bottom. In times of flood, water can be allowed to enter the canal through these gates with a high velocity so as to scour out the canal and remove silt. A short distance down the canal is constructed an escape for the surplus water that is admitted to the canal.

69. Typical Construction.—In Fig. 13 are shown the diversion dam *a* and regulator *b* built on the Boise River by the Reclamation Service. Over the spillway section, through which the water is shown discharging, is a roadway supported on trestle bents *c*. Here, the regulator *b* is in line with the diversion dam, but often it is at right angles to the dam. The flow of water into the canal *d* is controlled by the gates in the regulator.

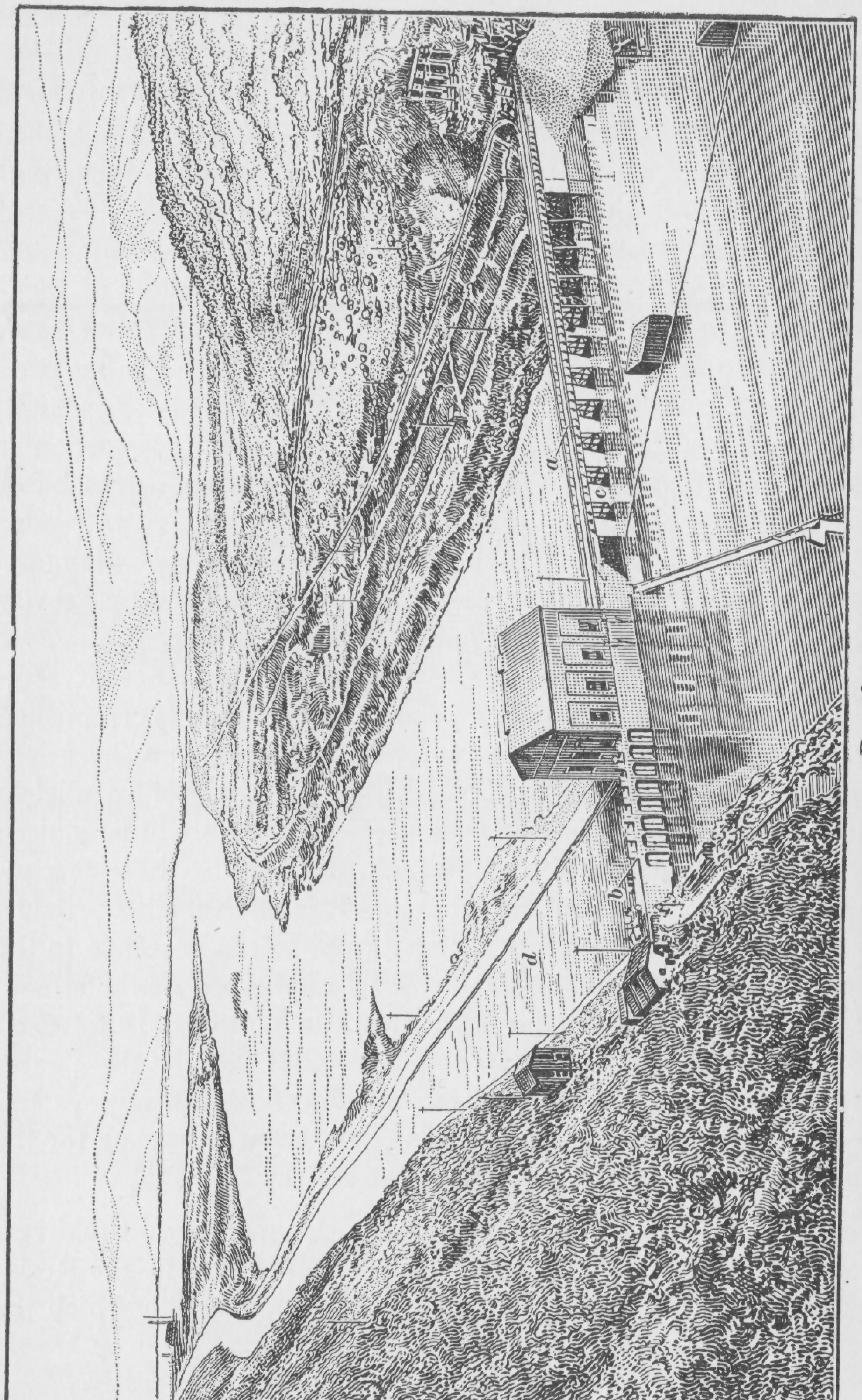

Fig. 13

70. Overflows.—When a large volume of water is conveyed in a flume or canal, there is always the possibility of overflow due to a sudden diminution of the draft without a corresponding reduction in the supply. This may be occasioned either by a shutting of the outlets or by some accidental obstruction to the flow; but, whatever the cause, an overflow unless directed through the proper channels is dangerous, particularly in the case of earthen canals.

Provision for overflow is usually made, in the case of earthen canals, by constructing spillways at intervals depending upon the gradient of the canal. Each spillway should be capable of discharging safely the maximum volume of water that can reach it when the entrance gates are wide open. As far as possible, spillways should be located above such potentially weak points as embankments and flumes, and opposite inlets for the admission of natural drainage into the canal. They should be constructed in a substantial manner to prevent damage from scour or wash.

71. Emptying Sluices.—In addition to the spillways, all canals should be provided with wasteways for emptying in case of an emergency, such as a break. Wasteways should be 10 to 20 miles apart, and generally may be located where the canal crosses natural drainage courses. Where such drainage courses are not available, lined channels should be provided to minimize damage to farm land. The bottom of the wasteway, where it leaves the canal, should be several feet lower in elevation than the canal bottom, so that the water will leave with a high velocity and will be drawn from both directions; hence the canal will be emptied rapidly. Wasteway gates should be ample in size and of a type that may be opened quickly; power-operated gates are used in many of the larger projects.

72. Drainage Crossings.—Where a canal crosses a stream or a natural drainage channel, special construction is generally required. If the canal is large and the amount of drainage water small, the water may be allowed to enter the canal. Otherwise, a flume or an inverted siphon may be constructed to carry the canal water over or under the stream. Where the relative ele-

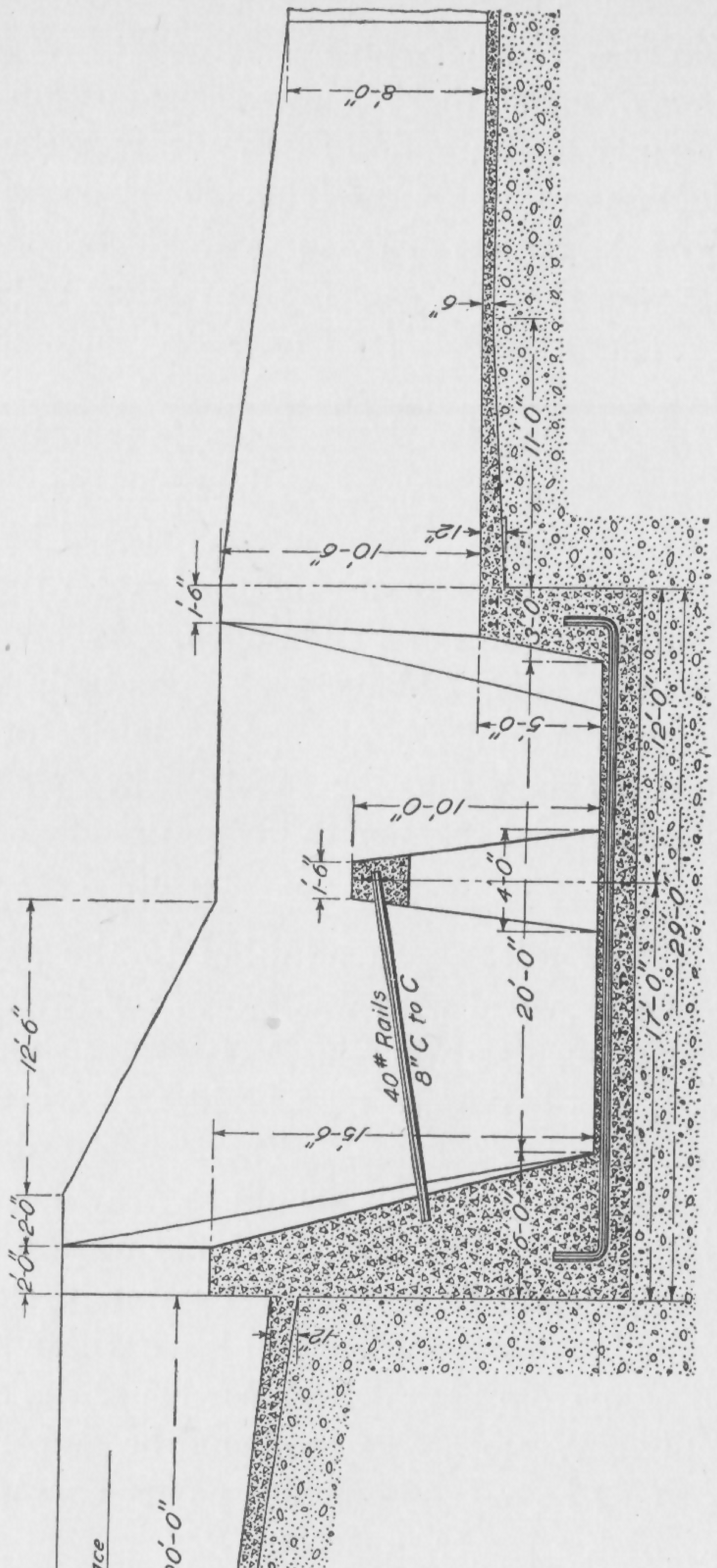

FIG. 14

vations permit, the canal may be carried over the drainage channel on an earthen embankment in which a culvert, usually of reinforced concrete, is built to allow for the passage of the drainage water. The waterway of a culvert should be ample to provide for the probable maximum storm discharge of the channel to be crossed.

73. Drops.—Where the country through which a canal runs has a greater fall than the safe grade of the canal, it is necessary to provide drops, which are structures designed to concentrate

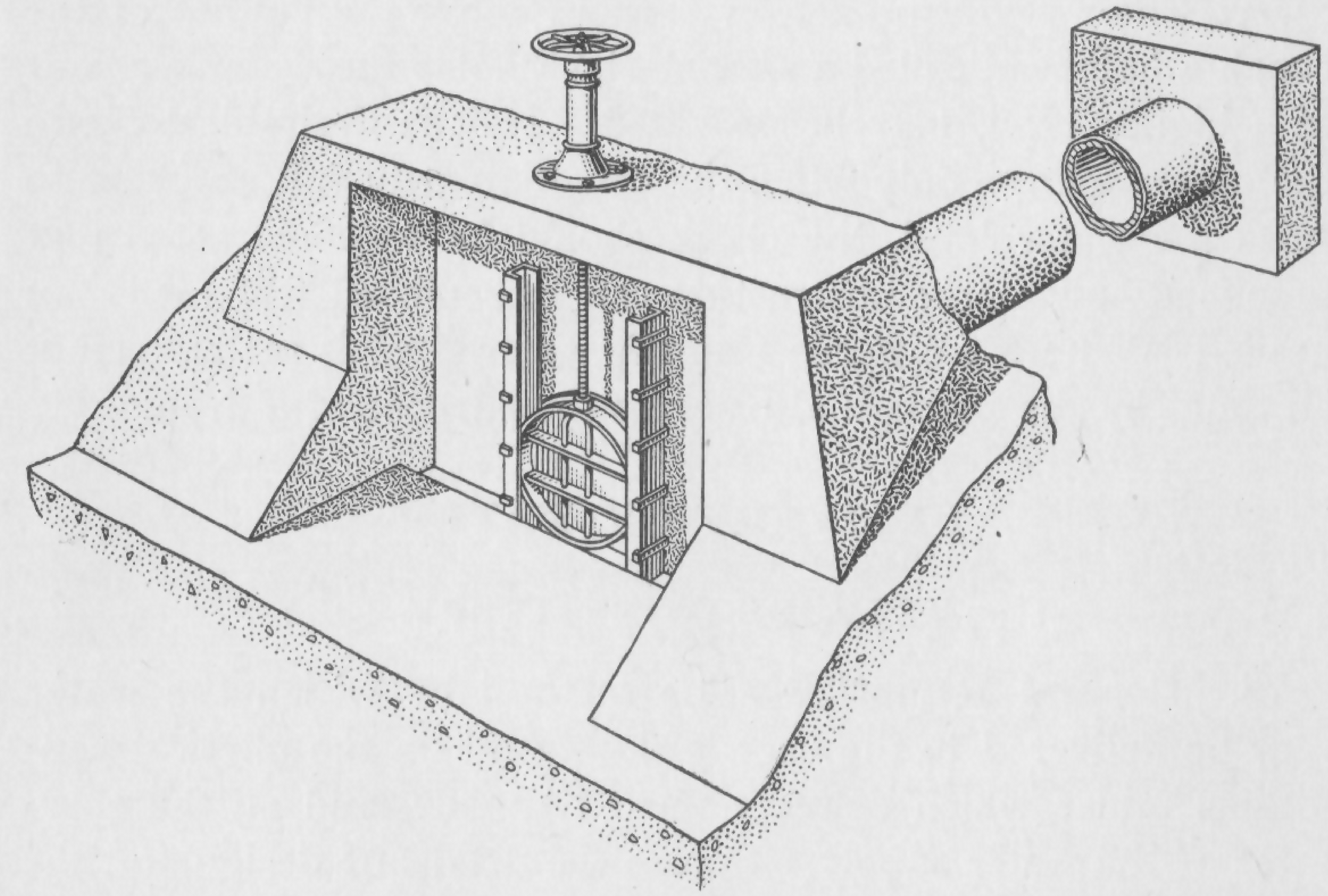

FIG. 15

the excess fall and to transfer the water from a higher to a lower level without injury to the canal. Drops may have a clear vertical fall, or the water may be caused to flow down an inclined pipe or chute.

A simple form of drop is shown in Fig. 14. The water at the upper level passes along a bed lined with concrete and over a concrete weir, which also acts as a retaining wall. It then drops into a water cushion whose depth is maintained at 5 feet by a concrete wall protecting the bed of the canal at the lower level. In order to reduce the erosive action still more, the fall of the water is broken by iron rails supported as shown.

74. Turnouts.—Wherever water is to be diverted from the main canal to a branch or where the canal is to be divided into two branches, a structure called a turnout is necessary. Turnouts should be designed to facilitate the passage of sediment and should therefore be at or near the bottom of the canal. Also, they should be carefully constructed to prevent leakage or percolation of water from the canal. A small turnout may consist of a pipe through the canal bank, with a gate at its inlet, as shown in Fig. 15. Larger turnouts are usually of reinforced concrete.

When it is required to increase the head on a turnout or to divert water into a branch or turnout above the bottom of the canal, a bulkhead called a *check,* which holds the water above it at a higher level, may be provided. If it is desirable to have the check operate only when the canal is partly full, in order to raise the water level, the check should be so constructed that practically all of the obstruction can be removed when it is not needed. Where a difference in the water level is maintained at all times by the check, it also serves the purpose of a drop.

DELIVERY OF WATER

METHODS OF DELIVERY

75. General Remarks.—The method by which the water may be delivered to the user is dependent on the physical conditions under which delivery must be made, and on the character of the water supply. Before the details of design and the capacity of an irrigation system are selected, a comprehensive study should be made of the moisture requirements of anticipated crops, effective water-holding capacity of the soil, and the factors of water supply and topography. There are three basic methods of delivering irrigation water: (1) continuous delivery, in which a stream of water is furnished to each irrigator at a uniform rate throughout the season; (2) rotation delivery, in which water is furnished to each of a group of irrigators in turn at regular intervals; (3) delivery on demand, in which the water is supplied to each individual when requested. The selection of the method to be used is important. Once a method is in effect, a change may be difficult, for the type of canal construction, its size, the size of delivery boxes and farm laterals, and the method of preparation of the land for irrigation may prove limiting factors.

76. Frequency and Duration of Delivery.—The frequency with which water is needed by the irrigator will depend upon his crop and soil requirements, an important factor being the capacity of the soil to retain moisture. With certain soils, it is feasible to irrigate heavily and at infrequent intervals, whereas with other soils, especially light soils which can hold less water in the root zone, frequent light applications are preferable, and even necessary. The duration of delivery is also a factor in determining the completeness of the irrigation, and the method of delivery should neither prolong unnecessarily nor cut short the period required.

77. Head or Stream.—The depth of the sheet of water that is applied to the ground is called the head or stream. The best head depends on many factors, of which the kind of soil, the type of crop, and the method of delivering the water are most important. Although the entire irrigation system must be designed to supply the water at the proper rate for the particular method of delivery chosen, there is usually some flexibility in the size of the head that may be delivered. An adequate head is large enough to cover the ground without excessive loss by deep percolation, yet not so large as to result in such a rapid movement of the water across the field that it is not available to the plants. Large heads require careful, and in many cases expensive, preparation of the land for their best utilization. On the other hand, serious losses of water may occur when inadequate heads are used on porous soils.

78. Continuous Delivery.—Continuous delivery has a distinct field of usefulness, but a relatively narrow one. Land lying on very steep slopes requires continuous delivery because of the necessity of using a very small head. This method is also advantageous on slopes where the soil is light and shallow, for small fruit and truck gardens, and for rice.

Continuous delivery is wasteful where soils are so light as to prevent a continuous stream from covering the ground without excessive loss of water due to deep percolation and, except where the water supply is limited, this method of handling water is inefficient.

79. **Rotation Delivery.**—In the rotation method of delivery, water is delivered by turns to various portions of the project and larger irrigation streams are available for the individual user than under the continuous method. The rate of delivery, the period of application, and the interval between applications are generally established according to local conditions, such as the type of soil on the project and the kind of crops. Thus, on one project in Southern California, where soils are heavy, the rate of delivery is 1 second-foot to every 10 acres for 24 hours each 15 days. On another project in Montana the delivery is 1 second-foot to each 30 acres for $3\frac{1}{2}$ days every other week, with the period of application doubled every fourth week for orchard tracts.

Rotation delivery not only puts the water supply to better use, but also gives better service to the individual. There appears to be a slight trend toward the use of rotation delivery, although there have been few projects in which the method of delivery, once selected, has been changed.

80. **Delivery on Demand.**—Obviously, delivery of water on demand is most satisfactory to the user and is best suited to his needs. In this method, water is delivered when requested by the user, in the quantities asked for and, within reasonable limits, with the desired size of the irrigation stream. Under proper conditions, delivery on demand eliminates waste in the application of water, but users do not always make the best use of the water supply, especially when there is only enough water to provide for the suitable irrigation of the entire project. Delivery on demand is often used in the early stages of an irrigation project, when the supply of water is somewhat in excess of the requirements.

81. **Organization for Water Delivery.**—The operation and maintenance of every irrigation system require office and field

forces which should be under the general charge of one responsible head, often known as *project manager*. In the case of the larger projects, it is usually found advantageous to have separate forces for the operation and maintenance; but, in the smaller projects, the same men take care of both.

A large irrigation system is usually divided into districts each of which is under the direction of a superintendent, usually known as *canal superintendent,* who is in charge of the delivery of water in his district and is responsible for the necessary maintenance work. He has under him canal riders or ditch tenders, watchmen, pump or gate operators, and other help.

For convenience in operation, the district is subdivided into sections each of which consists of a part of the canal and the laterals emanating from it and is under the care of a canal rider or ditch tender, who is responsible for the delivery of water to the users of that section. He also patrols the section regularly, reports conditions to the superintendent, and makes all minor repairs that are needed.

GAGING WATER FOR USERS

82. **Extent of Gaging.**—There are many reasons why the amount of water supplied to each user should be measured. Where the water supply is limited, the minimum amount of water required to irrigate the lands must be delivered, and this amount can be determined only through measurement. Excessive application of water is injurious, and it is very important to supply only the amount of water required for plant consumption. Moreover, it is but fair to everyone that each receives the water to which he is entitled, and no more. Charges for water should be based on the amount used, which it is always desirable to know. Nevertheless, it is not the general custom to measure the delivery of irrigation water to individual farms, except where the water supply is limited. This neglect is due, perhaps, very largely to the difficulty of securing dependable results without a considerable, and in some cases unjustifiable, cost. Some of the newer projects are provided with facilities for measuring the water used by individuals, but on few of the older projects has measuring been adopted. The justification for the installa-

tion of measuring devices depends largely on the value of the water.

83. Methods of Measurement.—While it is sometimes believed that the judgment of an experienced ditch tender is as accurate as an improperly used metering device, a real attempt is made in many places to measure the water supplied.

Rectangular and Cipolletti's trapezoidal weirs and submerged orifices are the devices most commonly used, but Venturi flumes and Foote water meters are also employed. Weirs are more generally suitable where the head is sufficient, and submerged orifices where there is less fall and the heads to be measured are smaller.

84. Measurement by Weirs.—The discharge of a weir can be readily calculated when the type of weir, the length of its crest, and the head or depth of water above the crest are known. Since the only variable factor for any particular weir is the head, it is customary, when weirs are used, to furnish ditch tenders with tables or charts showing the amount of discharge of the weir for certain depths over the crest.

85. Discharge From Orifice.—If the quantity of water to be measured is not too great, and the amount of debris in it is negligible, measurement by orifices is cheap and satisfactory. The orifice found most suitable for this work is the vertical standard rectangular orifice; the edges should be sharp and the up-stream faces vertical. The discharge of such an orifice may be found by the formula

$$Q = 4.89a\sqrt{h}$$

in which Q = discharge, in second-feet;
a = area of orifice, in square feet;
h = head on center of orifice, in feet.

Progress has been made in the development of circular pipes rated for use as submerged orifices.

86. Parshall Measuring Flume.—In the Parshall measuring flume, also known as the *improved Venturi flume,* a contraction is produced by placing a curved metal or concrete sheet

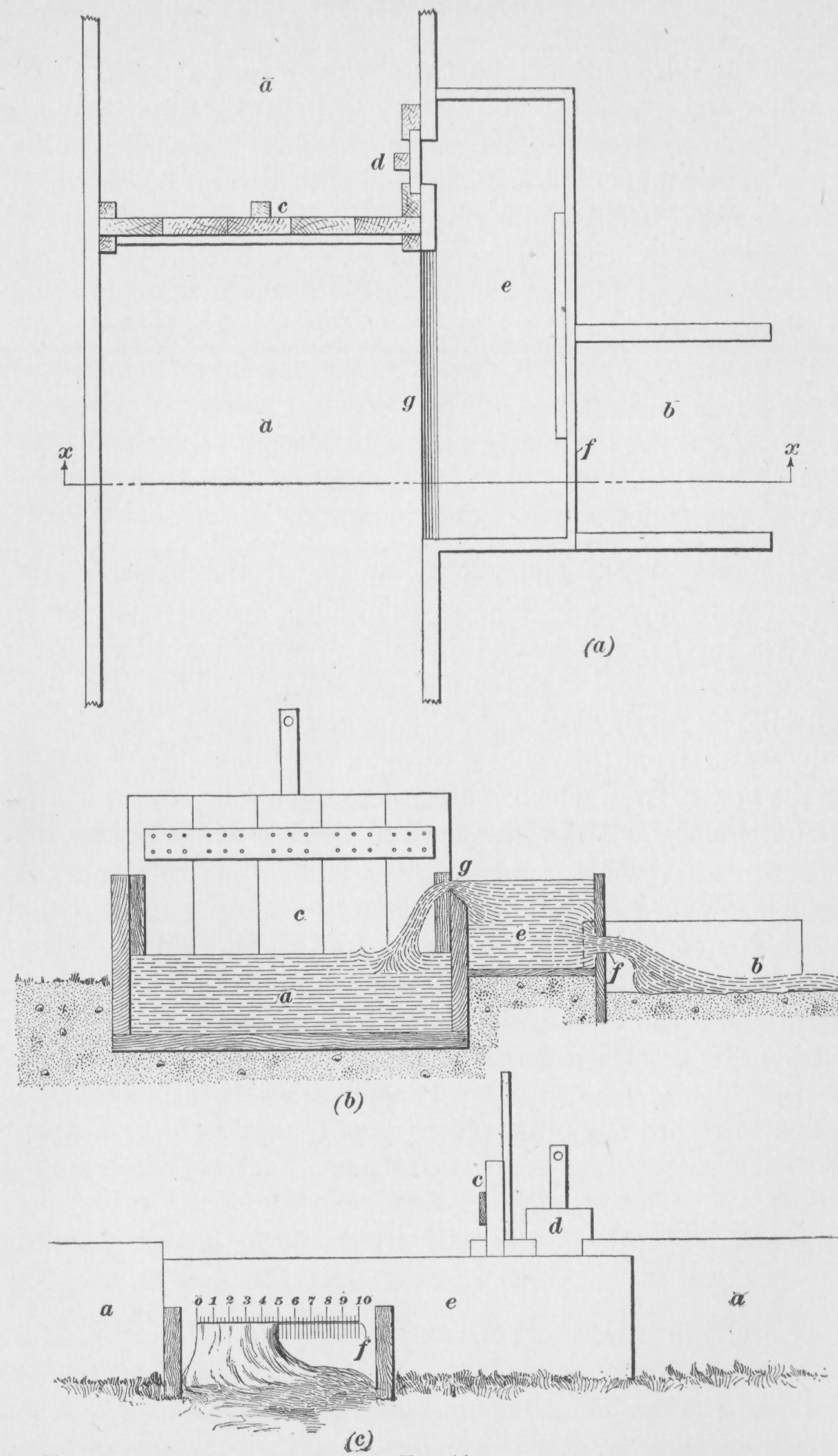

Fig. 16

between the sides of the flume, thereby depressing the surface of the water and increasing the velocity at the contracted sections. At the point of greatest contraction, the sheet is parallel to the bottom of the flume and is perforated with several holes, which allow the water to rise and form a still-water pond over the sheet. The difference in height between the water level at a point in the waterway near and above the measuring device and the level of the still-water pond, which is the difference between the pressure heads at the two places, is determined by means of gages, and the discharge is computed as for a Venturi meter. The use of the Parshall flume has increased for both main canals and individual delivery ditches. It is now recommended by a number of American states.

87. Foote Water Meter.—By means of the Foote water meter, shown in plan in Fig. 16 (*a*) and in cross-section along plane *x*-*x* in (*b*), any desired quantity of water can be diverted from the main canal *a* to the delivery ditch *b*. This diversion is effected by partly closing the sliding gate *c* so as to force some of the water from the canal *a* through the small sliding gate *d* into the box *e*, from which it enters the ditch *b* by means of the horizontal slot *f*. The water level in the box is limited by the height of the overflow *g*, the surplus water falling back into the canal *a* below the gate *c*, and the amount of water passing through the slot *f* is controlled by moving a gate that slides horizontally on the inside of the box. After a few trials, the openings of the gates *c* and *d* can be adjusted so that the proper level is maintained. The discharge is indicated, as shown in view (*c*), on a scale, which is here graduated in miner's inches but may be marked to record any other unit.

APPLICATION OF WATER TO GROUND

METHODS OF IRRIGATING LAND

88. Introduction.—The processes of collecting, storing, transporting, and delivering the water are all merely incidental to the task of getting the water on the land in order to produce crops. This is the most important operation, and is perhaps also the most complex. The problem is: Given a certain area of land, and a certain volume of water with which to irrigate it, how shall this water be applied to the ground so that every portion may receive a sufficient, but not excessive, degree of moisture, and so that no water shall be wasted? There are many methods of applying the water. The choice between them depends on many factors, including the kind of soil, its previous preparation, the slope of the ground surface, the kind of crops, the depth of the soil, and other important points.

89. Preparation of the Land for Irrigation.—It is preferable to prepare the land for irrigation after the main supply ditches, that is, laterals and sublaterals, have been built, rather than to locate and excavate the main ditches to suit land that has already been graded and leveled. However, field ditches should be located after the land is leveled. Crops are usually good or bad, depending on whether they have received the right amount of water at the right time; when the ground is so rough and uneven that water cannot be applied evenly, the effect is shown in the reduced yield. Therefore, proper preparation of the land is important, especially with such crops as alfalfa. The additional first cost of proper preparation is only $8 to $12 per acre and if the work is well done there is little after expense.

90. Before land is prepared for irrigation, a map showing contours with an interval of 6 to 12 inches should be drawn, and information regarding the nature of the soil and subsoil should be obtained. With these data, it is possible to secure

suitable locations for farm laterals, adjust the size and shape of the fields, and adopt a method of supplying water that will be most efficient under local conditions. The land is prepared by first removing vegetation, including brush, trees, and native grasses; next plowing deeply; and then bringing the surface, by means of graders or scrapers, to a smooth condition most suitable for irrigation. Formerly team-drawn equipment was used for this work, but in recent large projects, tractors and blade graders have been used on light grading, and rotary, fresno, or wheel graders on heavy work.

91. Types of Irrigation Methods.—Many ways of applying irrigation water are used in the United States, but they are all variations of four important methods, known as the flooding method, the furrow method, the check method, and the border method. The following material describing these methods is based largely upon information contained in Farmers' Bulletin No. 864 of the United States Department of Agriculture.

92. Irrigation by Flooding.—When a field is to be irrigated by flooding, small farm ditches called farm laterals or field laterals are run from the supply ditch across the field, as shown in Fig. 17, either before or after the seeds have been sown. Usually these field laterals are spaced 75 feet apart for grain and 90 feet apart for alfalfa, and have a grade of 3 or 4 inches to 100 feet. Sometimes, however, the laterals are made to extend down the steepest slope from the supply ditch. The water is allowed to escape from the field laterals through breaks in the banks, as indicated in the illustration. The ground should be evenly graded so that the water will flow in a thin sheet over the entire surface, the general direction of flow being shown by the arrows. An uneven surface causes waterlogging of the low places and insufficient irrigation in the high spots.

Although laborious, and not the most effective method of irrigation, flooding is quite generally used in many sections. It is cheap, and is fairly well adapted to certain local conditions, such as where the land is rolling and the good soil shallow. It is best used on small farms where the land to be irrigated is reasonably cheap, where the water is delivered in continuous streams or in

small heads for a length of time, and where grain and forage crops are to be raised.

93. Furrow Irrigation.—In the furrow method of irrigation, the field is divided into parts each of which is watered from a head ditch, and small ditches or furrows 2 to 4 feet apart are run from each head ditch at such an angle across the slope as to insure a gradual descent; the water is allowed to enter these fur-

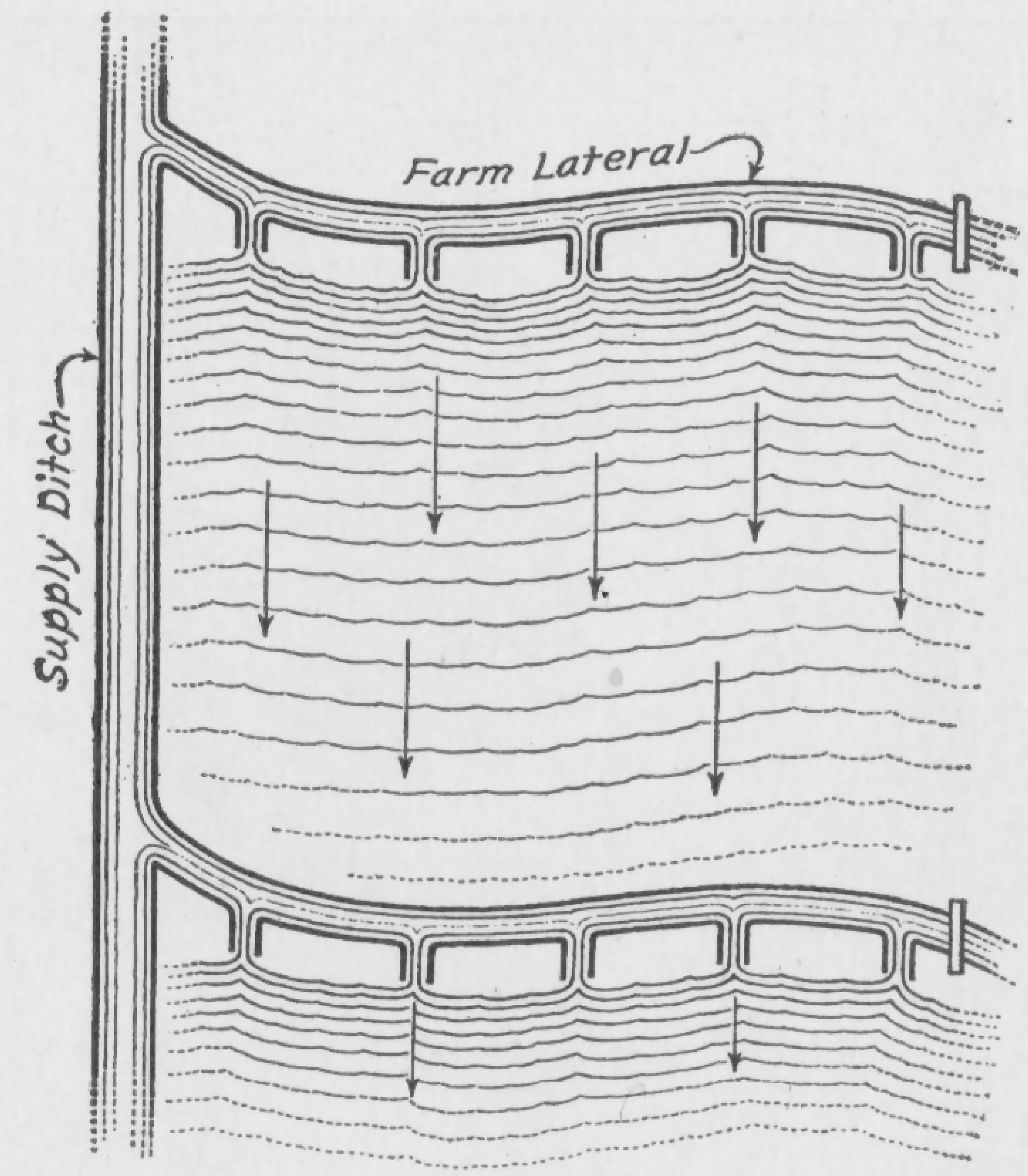

Fig. 17

rows and flow through them over the field. In porous, sandy soil, furrows should not be more than 300 feet long; in soils which absorb water less readily, they may be from 400 to 1,000 feet long.

A difficulty in this method is the division of the water in the head ditch somewhat equally among a large number of furrows. This is accomplished best by the use of small flumes, pipes, or spouts of uniform size leading from the head ditch to each furrow. The ditch should then be divided, by means of checks and drops, into a series of compartments, in each of which the water surface is level, and all spouts should be set at the same distance, about 2 or 3 inches, below the water level.

Nearly all crops that are planted in rows and cultivated are irrigated by means of furrows. Some important examples are potatoes, sugar beets, corn, cotton, melons, vegetables, and fruit. The cost of application of water is about the same by this method as by flooding from laterals.

94. Corrugation Method.—The corrugation method of irrigation, shown in Fig. 18, is a modification of the furrow method.

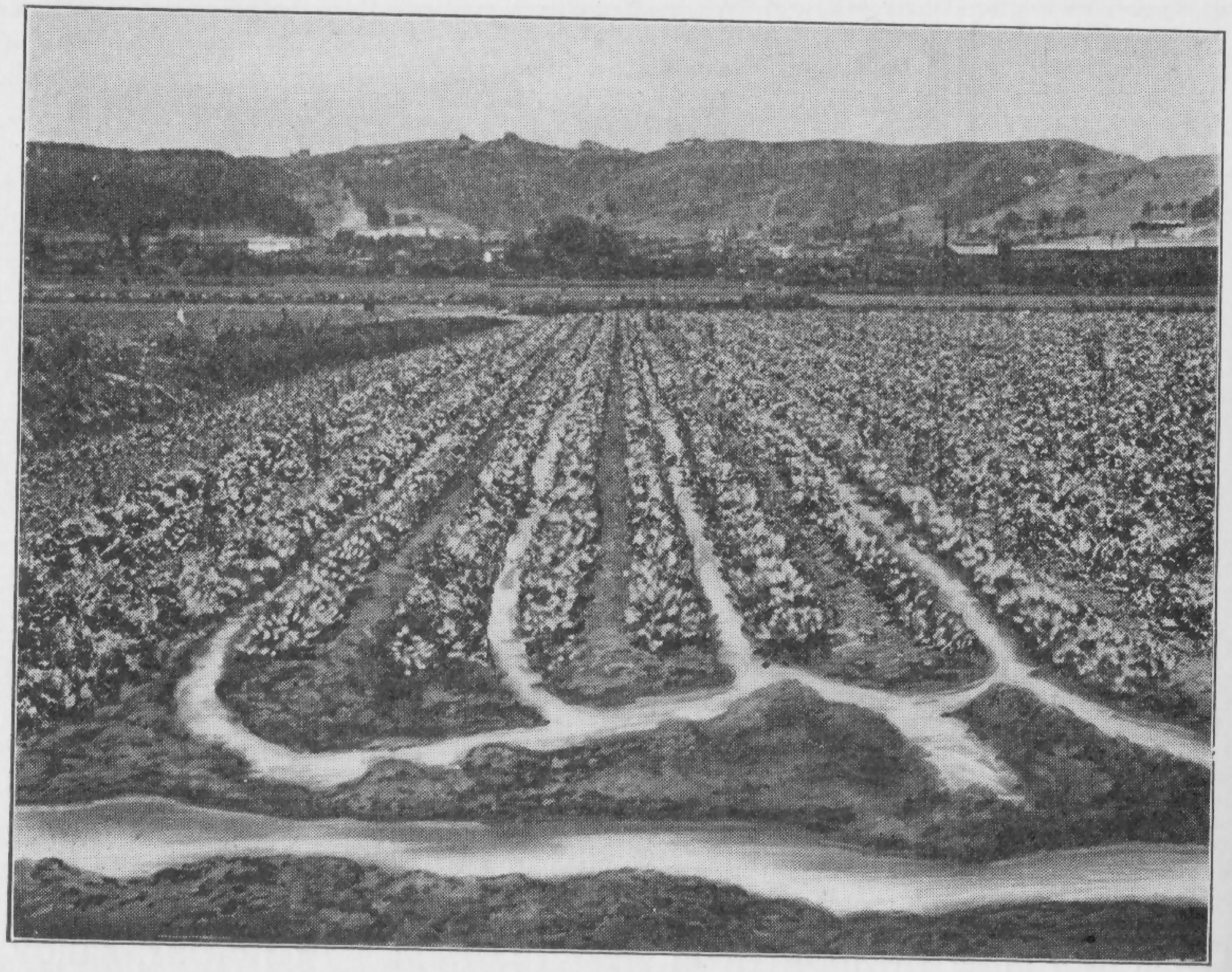

FIG. 18

The principle involved is that of subirrigation accomplished by allowing small streams of water to flow through a series of narrow, shallow furrows or corrugations long enough to permit the horizontal seepage from adjacent corrugations to meet and thus bring about a thorough wetting of the soil between the corrugations. The furrows are several inches deep and from 2 to 3 feet apart.

This method of irrigation is well adapted to the efficient application of water to steep or irregular slopes, where the farmer is required to use a small stream of water, or for new land which

has not yet been prepared thoroughly for irrigation. Although generally resulting in the use of far too much water, the corrugation method permits of a better control of soil moisture at critical times than do the methods that involve flooding the entire ground surface.

95. Irrigation With Sewage.—When sewage is used for irrigation instead of water, a modification of the furrow method is usually employed. However, sewage is utilized in this manner in only a few sections in the southwestern part of the United States, and in these cases it is employed because of its moisture value. This practice conflicts with the higher demands of sanitation. Moreover, sewage flows at all times, whereas irrigation is necessary for only a limited period, usually not more than 4 months a year, and to dispose successfully of the sewage at other seasons requires extensive treatment works.

96. Irrigation by Checks.—In the check method of irrigation, fields are divided into rectangular checks or plots, each comprising, as a rule, from $\frac{1}{2}$ acre to $1\frac{1}{2}$ acres. Around the margin of each check, a low embankment or levee is formed which retains the water until it has been absorbed by the soil. The use of this method is confined mainly to the growing of alfalfa.

In laying out rectangular checks, contour lines are run at intervals of 3 to about 6 inches, depending on the slope of the ground, and the rectangular checks are fitted into the spaces in such a way as to require the moving of the least possible volume of earth. With 3-inch contours on land which slopes 8 feet to the mile, the contour lines would be about 160 feet apart. On steeper slopes, this space is decreased and the difference in elevation between adjacent contours increased. Such checks cost more, but are convenient for farming operations. Land which slopes 50 feet or more to the mile is not suited for check irrigation.

A lateral ditch is built to carry water to each check or pair of checks. The capacity of these ditches should usually be about 10 second-feet. Each check should be provided with a gate of wood or concrete.

97. Border Irrigation.—In preparing land for border irrigation, it is divided by means of dikes into a number of parallel

strips, also called lands or beds, which extend down the steepest slope, as shown in Fig. 19. These dikes are from 5 to 8 inches high and have gentle side slopes, so that machinery can pass over the field with ease. The nature of the soil, the amount of water available, the slope of the land, and other similar factors determine the width and length of these strips. In sandy or gravelly soils, where water is absorbed readily, the length should not exceed 330 feet and the width 25 feet; in ordinary loamy soils the strips are usually 30 to 40 feet wide and 400 to 600 feet long;

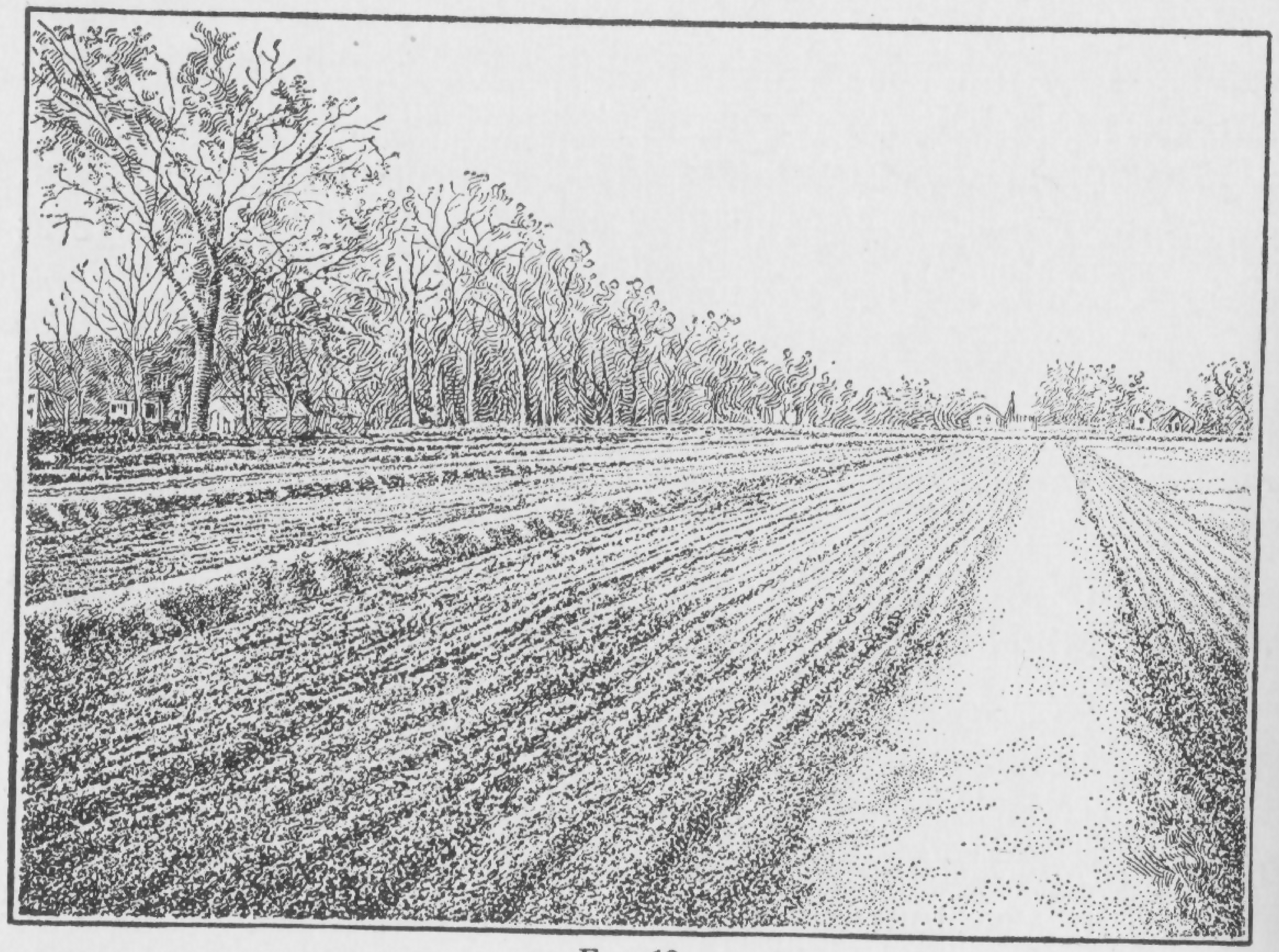

FIG. 19

in tight soils the width may be 50 feet and the length 1,000 feet. The most favorable soil for this method of irrigation is a free loam underlaid by a more or less impervious subsoil.

A smooth regular surface having a slope one way of about $2\frac{1}{2}$ inches per 100 feet may be regarded as most nearly ideal for border irrigation, although it is possible to make borders on slopes of 1 inch or less in 100 feet, and on steeper slopes up to 2 feet or more per 100 feet. The water is supplied from a head ditch located on a proper grade along the upper edge of the field. The amount of water to be used on a strip varies with the width

and the length of the strip and the type of soil from $\frac{1}{2}$ second-foot to 10 second-feet.

After the ground has been prepared, irrigation is accomplished by turning into the upper end of each strip the required head of water. This moves down the slope in a thin sheet, moistening the soil to a predetermined depth as it advances toward the lower end. The water is diverted into the strip by placing a canvas dam or permanent check in the head ditch, and opening the gate leading to the strip.

This method of irrigation is well adapted to the growing of alfalfa, all the grains, and clover and other forage crops; it may also be used with sugar beets, potatoes, and all rowed and cultivated crops by making a slight change in the borders. The cost of preparing land for this method of irrigation is low when the physical conditions are favorable. It is also usually possible to obtain a fair crop at a small cost by the use of temporary borders, which is a point of value in the first stages of settling a project. The corrugation method of irrigation may sometimes be used to advantage in conjunction with the border method as a means of spreading the water evenly over the border strips.

98. Subsoil Irrigation.—In subsoil irrigation, which is not used to any extent in the arid regions, but rather in truck farming in the East and South, a series of open-joint drains are laid 15 to 24 inches below the ground surface. The water is applied through this system of piping directly to the root area of the plants. Subsoil irrigation has the additional advantage of requiring a comparatively small amount of water, all of which is usefully employed.

99. Sprinkling.—Sprinkling water on the crops is another method employed almost solely in truck farming and nurseries. The construction and maintenance of the sprinkler system, which may be either overhead or underground, is costly, and is feasible only where crop value yields are high. This method has no application in arid regions for use on a large scale.

ALKALI AND WATERLOGGING

100. Cause of Trouble.—Experience has shown that the lack of drainage is the greatest single factor in rendering irrigated land unfit for use. Irrigation water applied to the surface of the ground passes downwards through it, dissolving and carrying with it various chemical elements contained in the soil. Some of the water usually reaches the water table and, unless it can escape to deep drainage channels, the ground-water level rises, and the dissolved salts are brought upwards. If this soil water is then brought by capillarity to the ground surface, and thence evaporated, the salts are left to accumulate, affecting many crops. Also, when too much water is used, the ground becomes waterlogged unless adequately drained. The United States Department of Agriculture has estimated that nearly 20 per cent. of the area that has been irrigated in the western states has been damaged either through waterlogging or the accumulation of salts. This fact emphasizes the importance of drainage as a necessity for most irrigated areas.

101. Alkali.—In irrigation practice, alkali is the term applied to any salt that is soluble in water. Locally such salts are classed as black alkali and white alkali, and these terms are commonly used, although they are not always strictly correct. The most important of the black alkali salts are sodium carbonate and sodium hydrate. Both of these are exceedingly destructive to plant growth, and in some cases when present in proportions of only .05 per cent. will render agriculture unprofitable. Of the white alkali, chlorides and sulphates of sodium, magnesium, and potassium are most common. These are less toxic to plants and are leached out more readily. Their presence in proportions of .5 to 1 per cent. is decidedly injurious to the soil, and half as much as this frequently causes considerable loss.

102. Preventive and Corrective Measures.—Measures designed to correct the troubles due to alkali and waterlogging may be either preventive or remedial, but there is actually no sharp line of demarcation between them. They include soil drainage by tile drains, open ditches, or pumps; reduction in the use of water; and the prevention of losses due to canal seepage.

103. Soil Drainage.—The use of open drains with flow lines below the level desired for the water table is common. With the use of machinery for cheap trenching, this method has been popular and quite widely adopted. Tile drains are employed to a considerable extent in certain sections, but the first cost is usually great enough to limit their use in large areas. Pumping from wells is rapidly becoming a recognized method of drainage, and has the great advantage of collecting water that can be used over again for irrigation. In fact, the employment of ground water for maintaining a supply of water during dry periods and in times of water shortage is a standard practice in many sections. Numerous large and important projects now depend to a certain extent on ground water to carry through dry cycles, and the design of a number of projects has provided that 10 to 30 per cent. of the water should be secured from the ground.

104. Reduction in Use of Water.—Control of the use of water and prevention of canal seepage are measures designed to reduce the amount of excess water in the soil. Efficient use of water is a difficult and complicated problem that requires a great deal of educational work in its solution. While irrigation of the soil to a depth of 5 feet is all that is ordinarily required, the amount of water that is needed to accomplish this cannot usually be stated, as it depends on the structure and texture of the soil, the temperature, and the presence of organic and mineral matter. Ordinarily, on average soils, water may be applied to a depth of 1 inch for each foot of depth to be moistened; but many soils require less water, $\frac{1}{2}$ inch per foot being sufficient in sandy soils. Thus, while there is some promise in the reduction of use of water, the immediate practical results in most places are small.

105. Prevention of Seepage Losses.—Losses from canals through seepage are very heavy in some cases. While such losses could be reduced very materially by lining the canals, it is often cheaper to suffer the losses from seepage than to pay for the cost of the lining. Accurate figures of losses are rare, but studies

made by the United States Reclamation Service showed a mean loss per day of more than 10 inches in depth over the wetted area. The losses in new canals that are unlined probably are in excess of one-third of the total amount of water carried and in many canals amount to one-half of the water. A loss in earthen canals of 1 second-foot per mile may be expected when the soil is unfavorable and the canal new.

Since the need for drainage is due to other factors also, lining of canals, while of value in preventing losses of water, cannot be expected to eliminate the need for drainage.

WATERING CROPS

COMMON PRACTICE IN APPLYING WATER

106. Introductory Remarks.—In order to design an irrigation system intelligently, the engineer should know something of how crops are raised, and the methods of handling the water in the process. Crop yields are best when the minimum amount of water and the maximum amount of cultivation are employed. More water is required during the first year than in subsequent years. Fixed rules cannot be applied to the growing of crops, but experience, judgment, and knowledge are necessary for success.

The material in the following articles is based largely on Bulletin 864 of the United States Department of Agriculture, which contains much practical information on irrigation farming.

107. Alfalfa.—Alfalfa is irrigated most commonly by being flooded from field laterals, but also may be irrigated by the use of checks, borders, or furrows. An ample supply of moisture should be provided before the seed is sown, thus avoiding the necessity for early irrigation; indeed, it is not generally desirable to irrigate again before the crop shades the ground, that is, when it is about a foot high. After the plants are well under way, one good irrigation for each crop will be sufficient. The time of irrigating differs in different sections. In some places it is the practice to irrigate before the crop is cut, while in other places the crop is cut first and the water applied to the stubble.

108. Grains.—Grain fields are usually irrigated by flooding from field laterals, but the border method has been gaining in favor. As a rule, the soil, as in the case of alfalfa, should contain enough moisture at seed time to nourish the crop until it shades the ground. A second irrigation is usually applied when the grain begins to head out. The amount of water required during the last three weeks of growth is small. In most irrigated areas, grain is used in rotation with alfalfa.

109. Potatoes.—Potatoes and other root crops are irrigated by furrows between the rows. These furrows should not be more than 600 feet long, and in light, sandy soils, a shorter distance may be necessary. Short furrows insure a more even distribution of water. If the field has been irrigated before planting, one heavy irrigation at the time the tubers begin to form may be sufficient, though in some cases from two to four waterings may be required. Shallow applications are not desirable, but the soil around the roots should be well supplied.

110. Fruit Trees.—The most common method of irrigating fruit trees is by means of furrows 500 or 600 feet long. Young trees are watered by a furrow on either side of the row, and as the trees grow older and larger, the number of furrows is increased until all the space between the rows is watered. The purpose to be attained is to train the roots outwards and downwards so as to enlarge the feeding zone. As in most other crops, shallow applications are undesirable, and the soil around and beneath the roots should be well watered.

WATER RIGHTS

111. Doctrine of Riparian Rights.—Under the doctrine of riparian rights, which is firmly grounded in our common law, streams and watercourses are preserved in their natural channels. The water is subject to use by the owners of the land bordering the stream, but it must be returned to the stream without substantial diminution in the volume or change in its character. This principle is opposed fundamentally to the necessities of irrigation, since in the arid regions crops cannot be grown unless water is diverted from the streams and applied to the soil. Con-

sequently, the law of riparian rights has been modified, the degree of modification varying in most states with the needs of agriculture. In the semihumid states, the common-law doctrine of riparian rights has been maintained, but laws have been passed which permit the diversion of water for irrigation. In the arid states, riparian rights have been superseded by the right of priority of appropriation for beneficial use.

112. Doctrine of Priority of Appropriation.—Under the doctrine of priority of appropriation, when two or more individuals or companies take water from the same stream, those whose rights were acquired last are the first to suffer when scarcity exists. When it became apparent that there was not enough water for the use of all, the principle was accepted that those who first made use of the water had the better right to continue that use. This doctrine of prior appropriation is, however, limited by the extent to which beneficial use is made of the water; that is, if the water is not put to a beneficial use, the right to the water may be lost.

In certain localities there has arisen also the system of prorating the water, or dividing it proportionately to the amount available, instead of strictly according to the priority of appropriation.

113. Rights Under Canal Company.—A canal company may have water rights, or it may act only as a common carrier, diverting, transporting, and delivering the water, and these acts do not constitute ownership of the water. The rights of an irrigator under a canal company are determined by his contract with the company. Some canals have plenty of water throughout the entire growing season; others carry a full volume during the flood season and a diminished volume during the remainder of the time; some fail to provide an adequate supply of water for the final irrigation; and others lose their entire supply after the spring floods have passed. Thus, it is evident that there are all kinds of rights to the use of waters, and the value of a water right often depends to a large extent on the nature of the stream from which it is taken, the priority of the right, and other factors.

SUMMARY ON IRRIGATION

114. Water Requirements.—Water is a necessity for plant growth, and where the rainfall is insufficient or does not occur during the growing seasons, irrigation is necessary for crop production. The amount of water required to maintain the degree of soil moisture that is necessary for crop growth varies with the crop, but depends even more on the soil, and above all on the skill of the irrigator; it is also dependent on the amount of rainfall during the growing season and other factors. Generally, it will be from about 1.5 to 3 acre-feet per acre of irrigated land, not including the losses encountered under average conditions of transportation of the water.

115. Procurement of Water.—Surface supplies are the most usual sources of water for irrigation, but ground waters are now being employed to an increasing extent, mainly in conjunction with surface supplies to carry through dry cycles or even to extend the irrigated areas. In some large projects, it has been estimated that from 10 to 30 per cent. of the water could be secured from the ground. Because rainfall is slight and storms are irregularly spaced, a thorough study of the run-off is a necessity in order to determine the amount of water that will be uniformly available for irrigation. Where the daily rate of run-off is insufficient for the demands of agriculture, storage reservoirs may be necessary, involving the construction of dams.

116. Transportation of Water.—Where the volume of water to be transported is relatively large, earthen or masonry-lined canals are generally employed for transportation of the water. The cost of linings, although considerable, may be more than compensated for by the saving in loss of water due to prevention of leakage and reduction of evaporation, and by the reduced cross-section of the canal that is allowable because of the greater velocity that may be employed.

Flumes are used to carry irrigation water across streams or low areas, and sometimes, under unusual conditions of topography, they are used for the entire waterway. They may be constructed of wood, metal, or concrete. Where the volume of flow is not too great, pipes of wood, metal, or concrete are sometimes used. They have the advantages that transmission losses are very small, and that problems of location are simplified.

117. Delivery of Water.—The water from the main canal is delivered through laterals and sublaterals to the network of farm laterals and ditches that bring it finally to the desired location. The canal and the main laterals are usually a part of the irrigation project, but the farm ditches are built by the landowners as needed. In some cases, water is measured as it is delivered to the farmer, but this is not the universal custom.

Water may be delivered continuously, by rotation, or on demand. The type of soil, the topography, and the character of crops are the principal factors in the choice of the method of delivery, but rotation delivery is most common and generally most suitable for average conditions.

118. Application of Water.—Several methods of applying the water to the ground are in use, the choice depending on the soil and topography, the water supply, and the crops. The most common methods are flooding from field laterals, generally suited to grain and alfalfa production; irrigation by furrows, which is well adapted to all crops that are planted in rows, as potatoes, corn, cotton, melons, vegetables, and fruit; the check method, which is applicable mainly to the growing of alfalfa; and the border method, which is especially suitable for growing alfalfa, grain, and clover, and may be used with a slight modification for rowed crops.

Proper preparation of the land before irrigating is important. It should be graded and smoothed to facilitate the application of the water.

119. Misuse and Loss of Water.—The quantity of water used by a plant forms but a part of that originally diverted for the purpose of irrigation. Evaporation and seepage from canals and ditches account for a considerable loss; seepage can be

reduced materially by lining the canals. Another important loss is that due to poor preparation of the ground. Neglect may also cause great loss, as when water is turned on and left to run for hours without attention. Too shallow and too frequent irrigation results in a waste of water, due to excessive evaporation when the surface is wetted and not cultivated afterwards.

In many sections, irrigation raises the ground water level and brings alkali to the surface or waterlogs the ground. The careful use of water and the employment of drainage may be necessary to prevent injury to crops from these causes.

120. Investigation of a Project.—Many of the large irrigation projects are composed of arid lands on which there are few or no settlers. Therefore, the development of a project may include not only the procurement of the water and the construction of the means whereby it may be delivered to the farmers, but also settling the project and bringing it into production. Lands must be purchased and arrangements for financing made. The unplanned and haphazard settlement and development of a large irrigation project may prove dangerous to the permanency and solidity of the project, and disastrous to the settlers. Therefore, a very careful investigation of any proposed irrigation project is necessary. This investigation should cover all phases of the problem, including the following: (*a*) the supply of water available for the project and the right to use as much of the water as may be needed; (*b*) the nature of the soil to be irrigated, its fertility and susceptibility to irrigation, the probable amount of water needed for agriculture, and the possibility of the need for drainage; (*c*) the plan of operation and maintenance to be used in the district; (*d*) the crops that may be raised, their market value, and the possibilities of marketing; (*e*) the methods of securing and financing settlers, and the probable rate of settlement and use of land; (*f*) the reasonable market value of the land when the project has been completed; and (*g*) the cost of the project.

From a consideration of these factors, a decision may be reached regarding the necessity or desirability of the proposed project.